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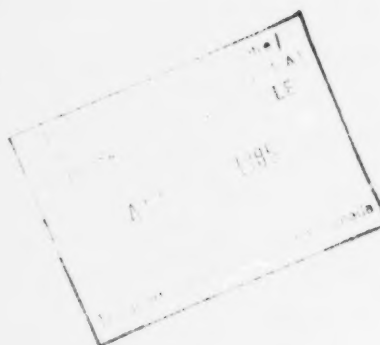
PREDICTING REQUIREMENTS
FOR SUBBALLAST THICKNESS

PROGRESS REPORT NO. 3

Measurements of Railroad Track Loads,
Roadbed Dynamic Deflections and Settlements
Period 1980-1982

Prepared For

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Un sommaire en français de
ce rapport est inclus avant
la Tables des Matières.

The contents of this report reflect the views of the contractor and not necessarily the official views or policies of the Transportation Development Centre (TDC) of Transport Canada.

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17 Abstract This project was commenced early in 1979 with the objective of developing construction depth design curves suitable for use with CN soils and track loading conditions. This progress report documents the 1981 roadbed elastic deflection measurements and the 1980-82 track-settlements. This report contains a preliminary guideline for depth of construction on Lake Edmonton clay. Granular depth requirements have been related to traffic density for comparable and acceptable levels of track surface maintenance. Relationships have also been established between wheel loading spectra and traffic density, roadbed settlement rates and elastic deflections/strains, track geometry car records and settlement rates, roadbed behaviour and track modulus. While work remains to be done in correlating many of the parameters (including the laboratory soils test work), it can be stated that data analysis techniques required for predicting future granular depth requirements from simple measurements on existing tracks have been clearly defined.			
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17 Résumé Ce programme a été lancé au début de 1979 dans le but d'élaborer des courbes de calcul de la profondeur à donner aux couches d'assise convenant aux plates-formes et aux conditions de services du CN. Le présent rapport traite des travaux de 1981 relatifs à la mesure de l'enfoncement élastique de la plate-forme et de ceux entre 1980 et 1982 relatifs à la mesure du degré de tassement. Ce rapport constitue un guide provisoire pour le calcul de la profondeur de la couche d'assise sur le sol argileux autour du lac Edmonton. La profondeur des couches granulaires a été corrélée à l'intensité du trafic en fonction du maintien de la voie à des niveaux de service comparables et acceptables. Des corrélations ont également été établies entre spectres de charges des roues et intensité du trafic; tassement de la plate-forme et rapport enfoncement élastique/déformations; évolution de la géométrie de voie et degrés de tassement; tenue de la plate-forme et module de voie. Bien que la corrélation entre ces divers paramètres, y compris les essais mécaniques de sol en laboratoire, reste à faire, il n'en demeure pas moins que les techniques d'analyse permettant de calculer la profondeur à donner à une couche d'assise à partir de simples mesures in situ des voies existantes sont maintenant clairement définies.		
18 Mots-clés Charge dynamique des voies ferrées, enfoncement élastique des plates-formes, tassement, déformation de l'infrastructure, profondeur de la sous-couche	19 Diffusion Le CDT dispose d'un nombre limité d'exemplaires	
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The Transportation Development Centre (TDC) of Transport Canada for their financial support.

EXECUTIVE SUMMARY

Work on this project was initiated early in 1979 with the objective of developing railroad track construction depth design curves for application on Lake Edmonton clays. One of the objectives was to determine if design techniques developed by British Rail Research were applicable for Canadian soils and traffic conditions.

This report presents a preliminary guideline for depth of construction on Lake Edmonton clay, and provides data necessary for estimating future ballast quantities required to keep pace with increasing traffic densities.

While several pitfalls and blind alleys were encountered during the four-year study, the work did keep on track. Important measurement parameters have been clearly identified and simple measurement techniques have been developed. The answers for future track construction and upgrading requirements for present lines lie in the measurement and clear definition of existing track performance. Lack of good field measured data is the primary reason why so many roadbed design theories have never received acceptance. Computers will not provide facts - they can only regurgitate information, and if the initial input was inadequate, the output will also be inadequate. This has been the case in the past.

Any future funding for expansion of this project should be devoted primarily to field measurements. Development of correlating theories may or may not be of benefit.

SOMMAIRE TECHNIQUE

Ce programme, lancé au début de 1979, visait l'élaboration de courbes de calcul pour la détermination de la profondeur à donner aux couches d'assise des voies ferrées posées sur le sol argileux autour du lac Edmonton. Un des objectifs du programme a été de vérifier si les calculs élaborées par le bureau de recherches de la British Rail étaient applicables aux sols et aux conditions de service du Canada.

Le présent rapport constitue un guide provisoire pour le calcul de la profondeur de la couche d'assise sur le sol argileux autour du lac Edmonton, de plus, il fournit les données nécessaires à l'estimation des quantités de ballast devant être mises en oeuvre pour faire face à l'accroissement prévu des trafics.

Malgré les erreurs et les culs-de-sac des quatre dernières années, les travaux ont continué sans désespérer. Ils ont débouché sur la mise en évidence d'importants paramètres de calcul et la mise au point de techniques de mesure aisées à appliquer. Le calcul des voies nouvelles et la définition des besoins d'entretien des voies existantes doivent reposer sur une détermination et une définition exactes de la tenue des voies existantes. C'est l'absence d'observations fiables in situ qui explique en grande partie le rejet d'un si grand nombre d'hypothèses de calcul. En effet, les ordinateurs ne créent pas de données, ils ne font que les restituer. Si l'information de départ est insuffisante, l'information restituée ne le sera pas moins. Telle a été l'expérience du passé.

C'est pourquoi, toute mise de fonds pour l'extension du présent programme devra être consacrée d'abord aux observations in situ. L'intérêt d'hypothèses de corrélation demeure incertain.

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1. INTRODUCTION

This is the third Report documenting track measurements from a research program initiated in 1979. The original objective of the work was to determine railway track granular depth requirements by means of correlating roadbed elastic deflections with subgrade soil properties. CN Research measured roadbed behaviour at twelve Edmonton area test sites with various granular depths in May of 1979. McGill University Geotechnical Research Centre was retained to conduct soils tests and provide computer capability for any required theoretical analyses. Correlations were obtained between tie and subgrade elastic deflections and depth of granular but there was no clear-cut method of determining which elastic deflection levels were acceptable and which were not.

In 1980, roadbed elastic deflections were measured at three Edmonton area sites and instrumentation was now geared for determination of strain gradients within the roadbed. In addition, subgrade settlement plates were installed at five Edmonton area sites in an effort to establish track deterioration rates. Track settlement plates were also installed in two Montreal area sites and roadbed elastic deflections were recorded at one of these sites.

All settlement plate measurements were continued throughout 1981 and 1982, and a series of roadbed elastic deflection measurements were taken at one of the Montreal area sites during 1981.

This Report presents information from the 1981 roadbed elastic deflection measurements and the 1980-82 track settlements. All of the CN field measurement work must still be correlated with the McGill laboratory work, and it is to be hoped that a final close-out Report containing validated granular depth design charts can be issued in early 1984.

Work on this project has suffered somewhat from the Canadian recession and the general drop in employment levels. While the authors would have preferred more comprehensive work in a shorter time frame, such was not possible. It is felt, however, that a few well documented field measurements are worth a dozen unvalidated theories, and the information presented herein may be utilized by others in possibly expanding upon the work. There is a learning curve in the development of new concepts, and it takes a while to weed out the interesting but useless information, and to establish the important critical parameters. There is now a clear understanding of just what those critical parameters are.

All previously published Reports on this work have been listed in the Bibliography.

2. JOLIETTE SUBDIVISION ROADBED ELASTIC DEFLECTIONS

2.1 Instrumentation Set-Up

The instrumentation set-up used for measurement of roadbed elastic deflections and related parameters is shown in Exhibit No. 1. Four cones with rod and casing were driven to the desired level into the roadbed. The rods were then tapped down an extra 1/4 in. to bed the cones and free them from the casings. The vertical displacements of these cones were then measured with transducers clamped by brackets to a 10 ft. bench mark rod. A strain gauge shear circuit was applied to the rail to measure vertical dynamic wheel loads, and two instrumented tieplates were used to measure vertical tie loadings.

The cones were driven in August of 1980 and were left in place for all the 1980 and 1981 dynamic measurements. This relatively simple installation provided an excellent means for deflection measurement of undisturbed roadbed at depth.

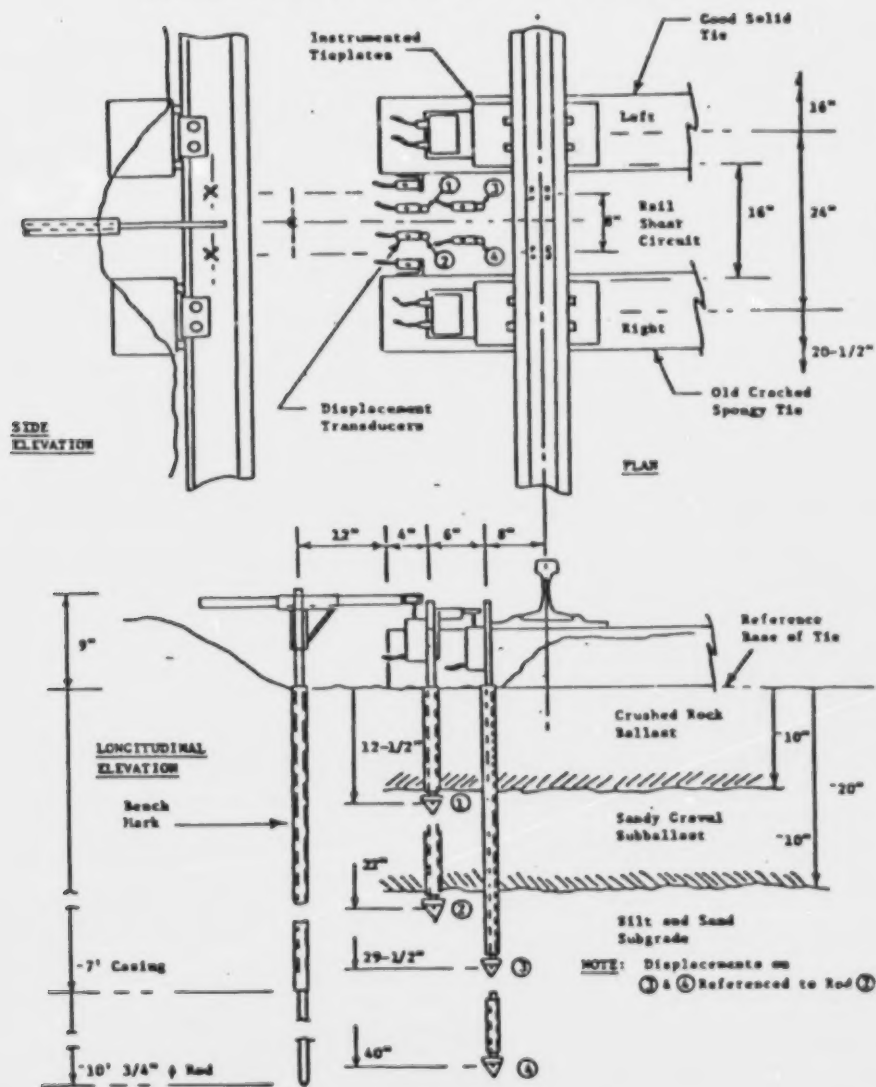
The general site plan for Site No. 18 at Mile 118.2 Joliette Subdivision is shown in Exhibit No. 2. In addition to the instrumentation described above, a total of 20 subgrade settlement plates were installed over a 100 ft. length of track. These 6 in. square plates with 1/2 in. diameter rod welded in center were placed on top of the subgrade (granular layer was dug out for installation), plastic tubing was placed around the rods, and the granular was backfilled. The rods extended above the top of ballast, and at frequent intervals over a 3-year period, elevations were measured with precise level with reading accuracy of ± 0.01 mm. Elevations on top of rail head were also measured at these 20 stations in order to determine roadbed total settlements.

Similar settlement plates were installed in 1980 at five Edmonton area test sites.

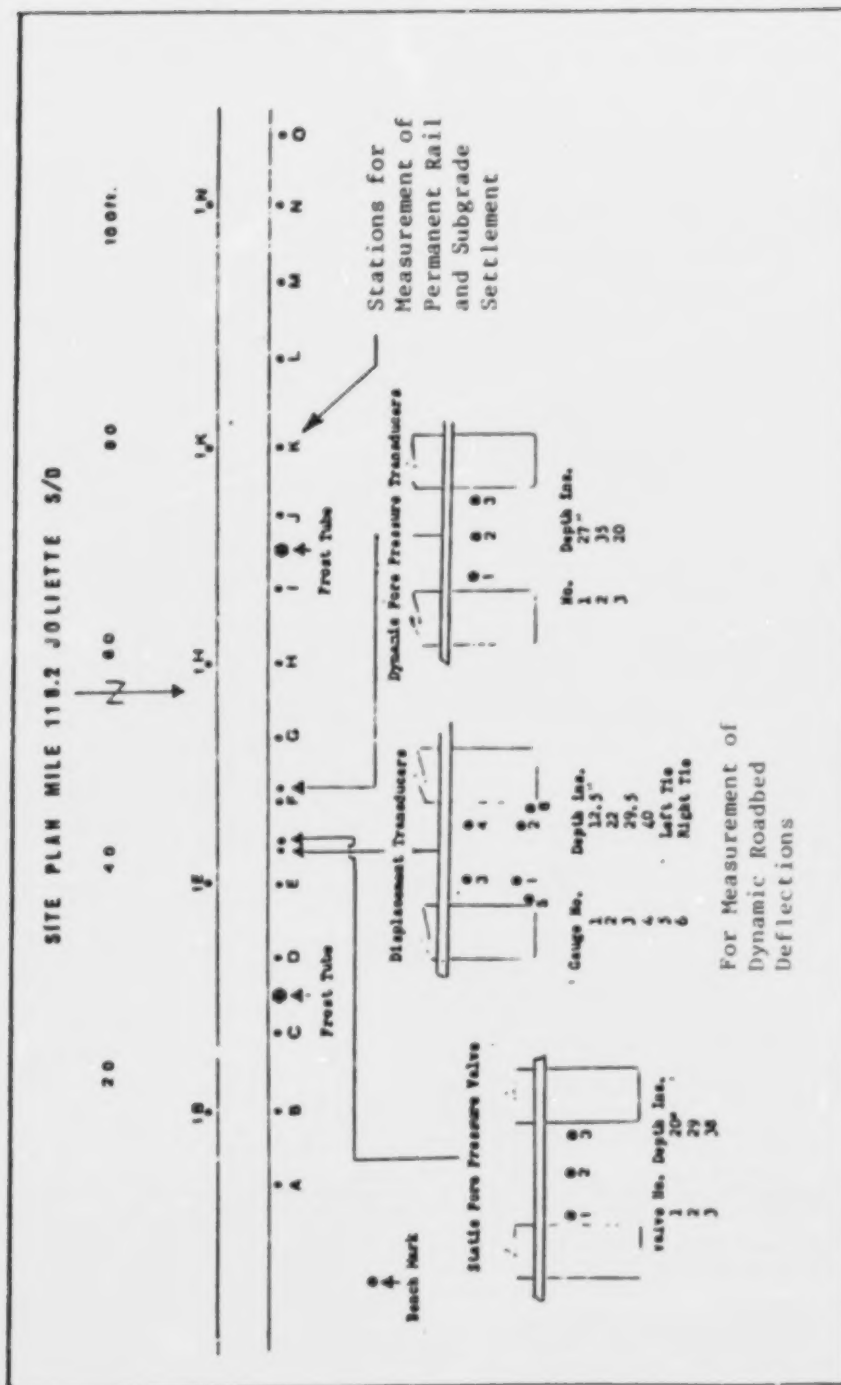
Two frost tubes were also installed at the Joliette site. These sealed plastic tubes, containing a liquid which changed colour when frozen, were dropped into a capped casing. A simple tape measurement gave frost lines on the tubes (there was a tendency for the tubes to freeze into the casings and it was not always possible to remove the tubes for measurement).

Subgrade soil static pore pressure valves and dynamic pore pressure transducers were also installed. Difficulty was encountered in getting good measurements, and none of the results have been presented in this report. This is one phase of the study that required more intensive work, and it is unfortunate that no useful information was obtained.

INSTRUMENTATION SET-UP, MILE 118.2 JOLIETTE SUB., AUG. 1980



SITE NO. 18 - INSTRUMENTATION SET-UP



2.2 Train Record Log

The train record log for the dynamic roadbed deflection measurements taken during 1981 is given in Table No. 1. A total of thirteen sets of measurements were taken, starting in March before the spring thaw and running right through until late November.

2.3 Normalized Roadbed Deflections

Sample plots of roadbed dynamic deflection vs. wheel dynamic vertical load are given in Exhibits No. 3 to 6. These deflections were measured with the cones embedded in the roadbed.

- Cone 1 - 12-1/2 in. below tie base, within granular
- Cone 2 - 22 in. below tie base, at top of subgrade
- Cone 3 - 29-1/2 in. below tie base, within subgrade
- Cone 4 - 40 in. below tie base, within subgrade

Strip charts were manually read and dozens of such graphs were tediously plotted. The scatter in points was horrendous, and straight lines were fitted by eye because it was not felt that rigorously fitted lines would be any more meaningful. These four graphs clearly illustrate the variable relationship between roadbed deflection and load, and explain why so many roadbed design theories remain unvalidated. The roadbed defies linear or non-linear theoretical analysis with shotgun blasts of data. Vehicles with variable axle spacings moving at speed respond dynamically to the track surface roughness in a complex and variable fashion. The more complex a design theory, the less chance it has of ever being validated. It is felt that field measurements should first be taken in statistical strength and then empirical or theoretical analyses applied. It has often proven useless in the past to develop a complicated theory and then try and validate it through a few isolated measurements. These theories usually end up gathering dust on some shelf because no railroad track design engineer has the time or patience to play with them. In effect they represent misspent funds. Money should be spent in taking field measurements because the answers lie in the roadbeds and will not be found in computers.

Recognizing the scatter of data in the load-deflection plots, the roadbed deflections have nevertheless been normalized per kip of wheel load. These normalized deflections for the four depths of measurement have been presented in Table No. 2. No semblance of accuracy can be claimed, but it should be stated that deflections were measured to $\pm .001$ in. and wheel loads to ± 1 kip. The normalized deflections ranged from 0.0013 in./kip at 12-1/2 in. below tie base to 0.0002 in./kip at 40 in. below tie base. These deflections are exceedingly small in an absolute sense and point out another reason why theories often remain unvalidated - the requirement for extreme precision instrumentation to function in a very rugged environment.

TABLE NO. 1

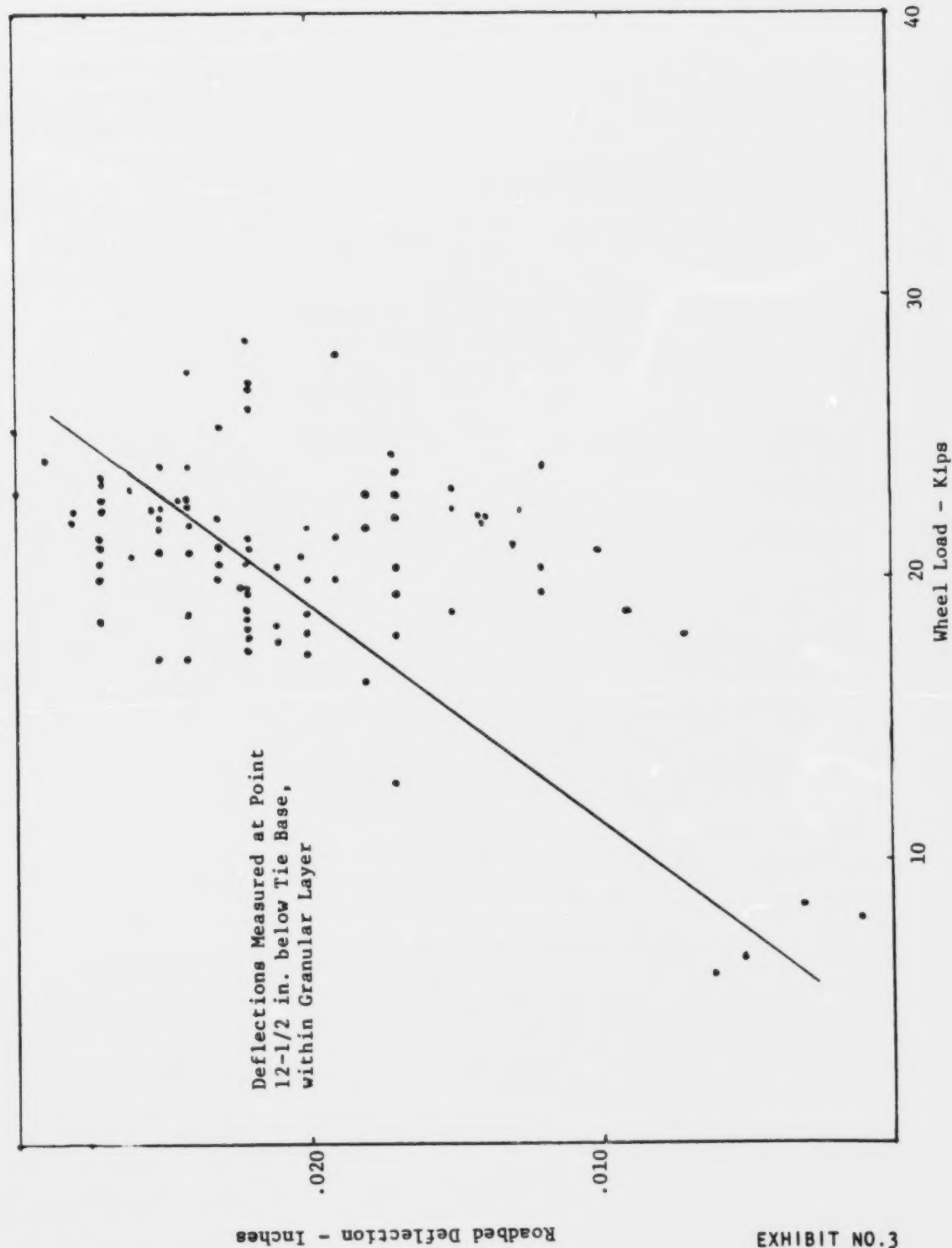
TRAIN RECORD LOG, SITE NO. 18, MILE 118.2 JOLIETTE SUB.

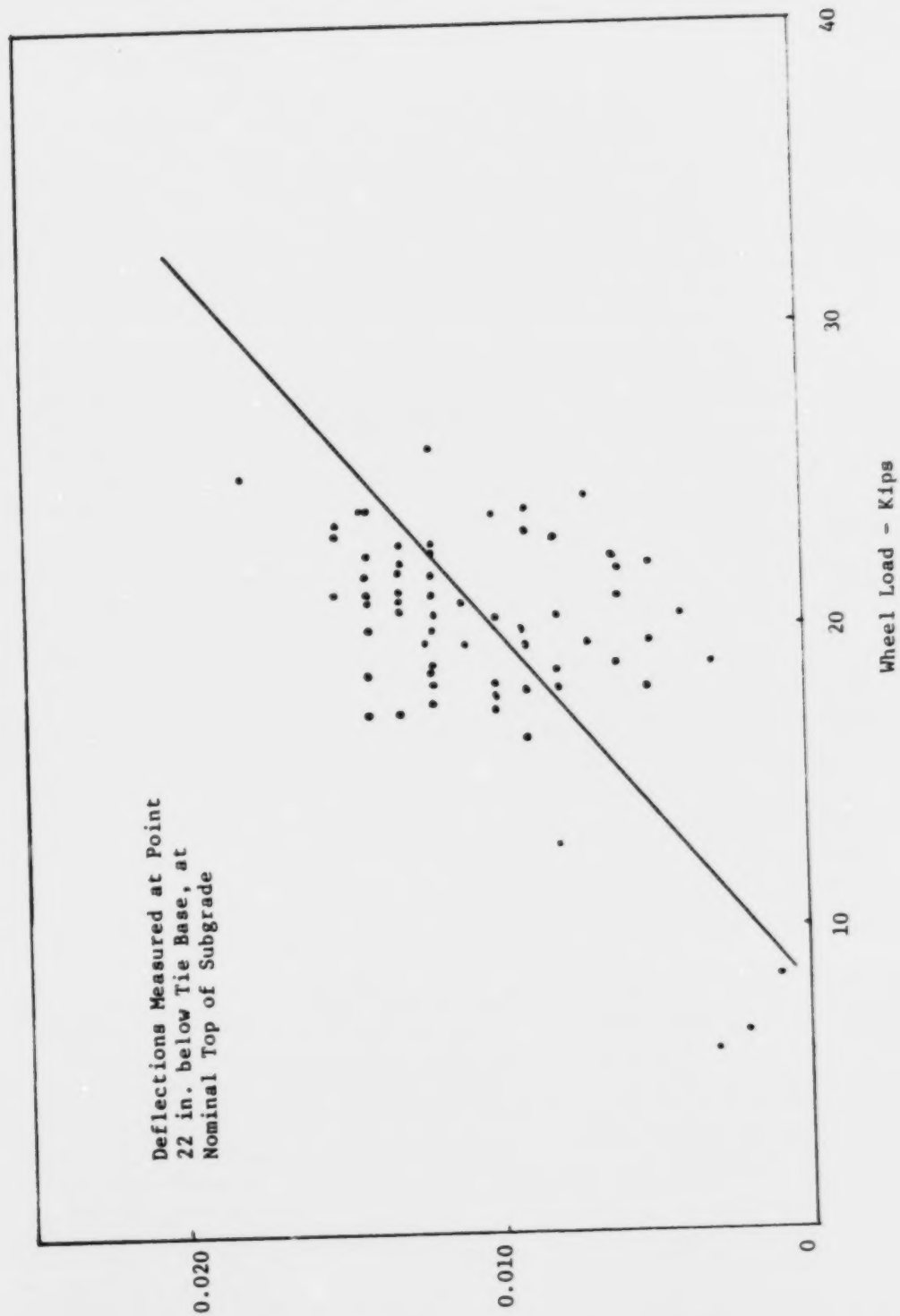
DATE	RECORD NO.	DIRECTION, TYPE OF TRAIN	NO. OF AXLES	COMMENTS	
MAR. 26/81	1	EAST FREIGHT		NO WHEEL LOAD	ROADBED FROZEN SOLID
	2	WEST PASSENGER	22		
	3	WEST FREIGHT	424		
APR. 1/81	4	PASSENGER FREIGHT	20	NOISY SIGNALS	ROADBED FROZEN SOLID
	5		444		
	6				
APR. 8/81	8	EAST MAY FREIGHT	36		ROADBED FROZEN SOLID
	9	EAST PASSENGER	16		
	10	EAST MIXED FREIGHT	400		
	11	WEST MAY FREIGHT	60		
APR. 23/81	12	EAST PASSENGER	16	NO WHEEL LOAD NO WHEEL LOAD	ROADBED PARTIALLY THAWED
	13	EAST FREIGHT	52		
	14	EAST PASSENGER	22		
	15	EAST FREIGHT	60		
	16	FREIGHT	446		
MAY 7/81	17	EAST PASSENGER	18		ROADBED THAWED
	18	EAST MAY FREIGHT	96		
	19	WEST MAY FREIGHT	96		
MAY 26/81	20	WEST FREIGHT	420	NO WHEEL LOAD FRONT 66 AXLES	
	21	WEST PASSENGER	28		
	22	EAST PASSENGER	18		
	23	EAST FREIGHT	224		
JUNE 18/81	24	WEST PASSENGER	28	NO WHEEL LOAD	
	25	EAST FREIGHT	404	NO WHEEL LOAD	
	26	EAST PASSENGER	26	NO WHEEL LOAD	
	27	EAST MAY FREIGHT	80	NO WHEEL LOAD	
	28	WEST FREIGHT	88	NO WHEEL LOAD	
JUNE 23/81	29	EAST UNIT OIL TRAIN	164	NO WHEEL LOAD	
	30	EAST BALLAST TRAIN	116	NO WHEEL LOAD	
	31	WEST PASSENGER	40	NO WHEEL LOAD	
	32	EAST PASSENGER	24	NO WHEEL LOAD	
	33	EAST FREIGHT	412	NO WHEEL LOAD	
	34	EAST FREIGHT	60	NO WHEEL LOAD	
SEPT. 25/81	35	WEST PASSENGER	34	NO WHEEL LOAD	
	36	EAST PASSENGER	18	NO WHEEL LOAD	
	37	EAST FREIGHT		NO WHEEL LOAD	
	38	EAST WORK TRAIN	26	NO WHEEL LOAD	
OCT. 8/81	39	WEST PASSENGER	28		
	40	EAST PASSENGER	18		
	41	EAST FREIGHT	32		
	42	EAST FREIGHT	412		
	43	WEST FREIGHT	92		
OCT. 21/81	44	WEST FREIGHT	464		
	45	WEST PASSENGER	40		
	46	EAST PASSENGER	12		
	47	WEST FREIGHT	64		
NOV. 12/81	48	WEST FREIGHT	432		
	49	EAST FREIGHT	100		
	50	EAST PASSENGER	18		
NOV. 26/81	51	EAST FREIGHT	48		
	52	WEST FREIGHT	480		

ROADBED DEFLECTION vs. WHEEL LOAD

SUBGRADE 1 TRAIN FREIGHT RECORD NO. 20, MAY 26/81

SITE NO. 18 MILE 118.2 JOLIETTE SUB.



ROADBED DEFLECTION vs. WHEEL LOADSUBGRADE 2 TRAIN FREIGHT RECORD NO. 20, MAY 26/81SITE NO. 18 MILE 118.2 JOLIETTE SUB.

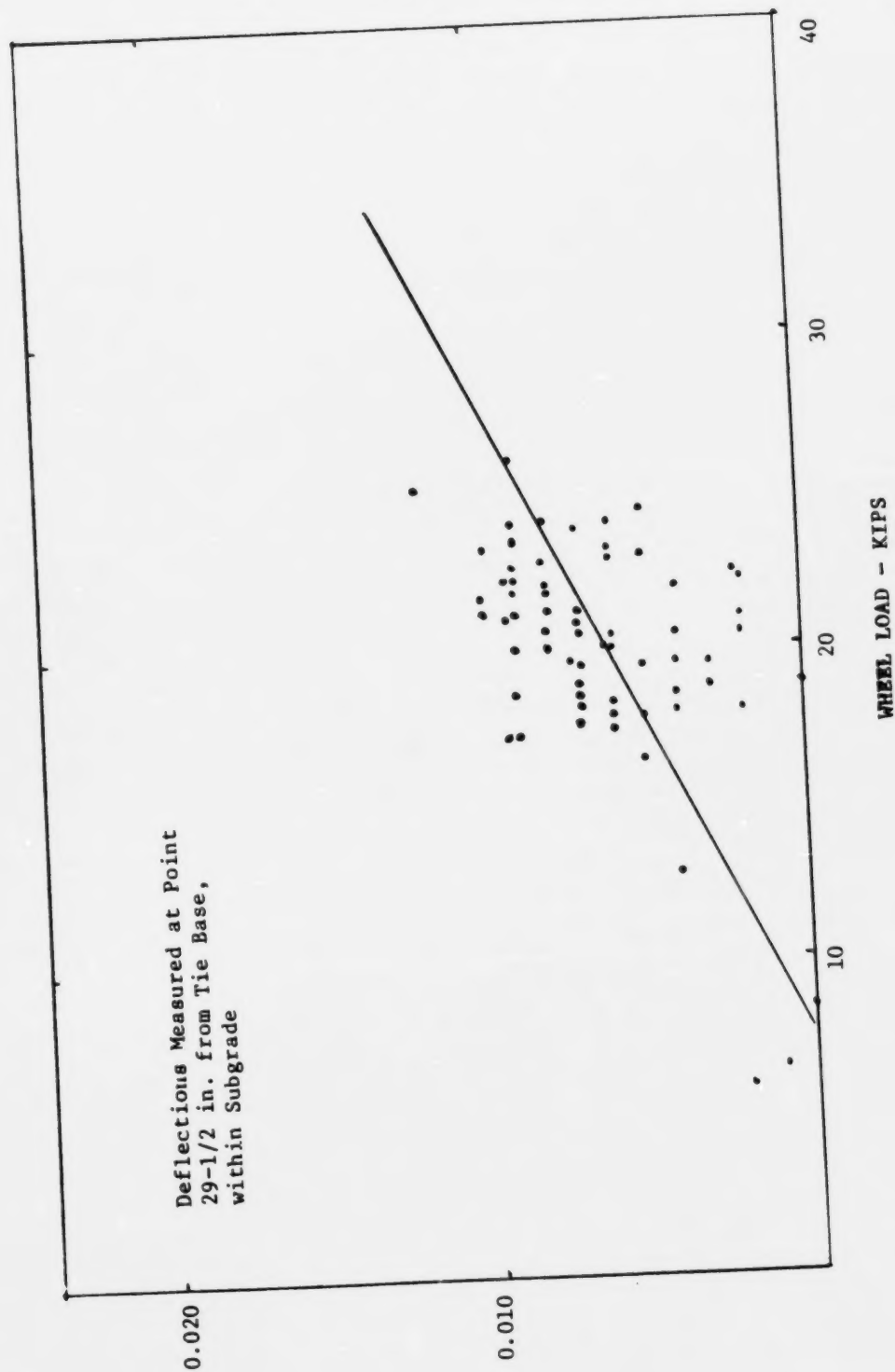
Roadbed Deflection - Inches

EXHIBIT NO. 4

ROADBED DEFLECTION vs. WHEEL LOAD

SUBGRADE 3 TRAIN FREIGHT RECORD NO. 20, MAY 26/81

SITE NO. 18 MILE 118.2 JOLIETTE SUB.



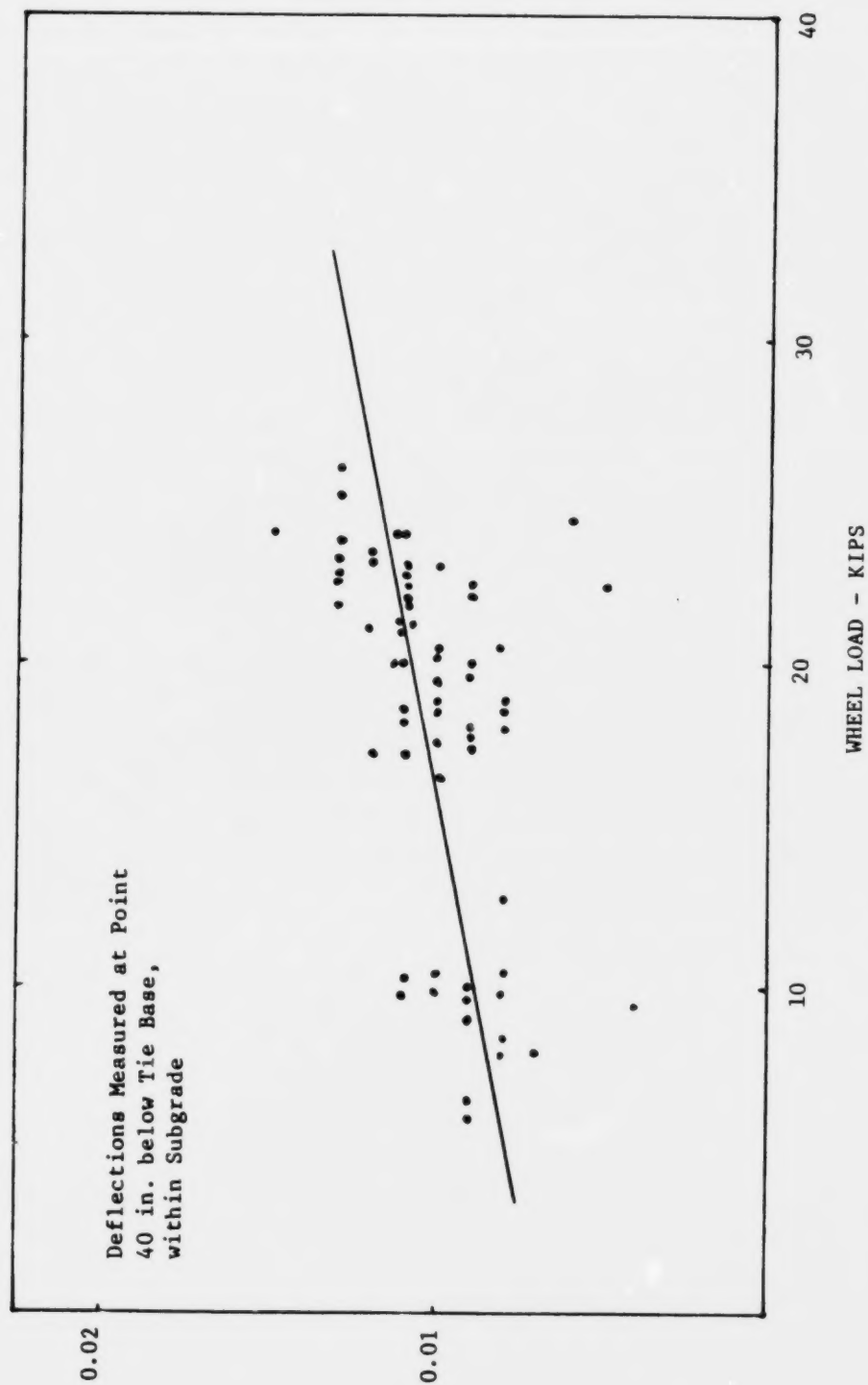
ROADBED DEFLECTION - INCHES

EXHIBIT NO. 5

ROADBED DEFLECTION vs. WHEEL LOAD

SUBGRADE 4 TRAIN FREIGHT RECORD NO. 20, MAY 26/81

SITE NO. 18 MILE 118.2 JOLIETTE SUB.



9.0N 118HX3

ROADBED DEFLECTION - INCHES

TABLE NO. 2
NORMALIZED ROADBED DEFLECTION (COMPLIANCE) VALUES
IN./KIP WHEEL LOAD
SITE NO. 18 MILE 118.2 JOLIETTE SUB.

Date	Deflection/Wheel Load (In./Kip)			
	Subgrade 1	Subgrade 2	Subgrade 3	Subgrade 4
Mar. 26/81	0.00059	0.00058	0.00058	0.00061
Apr. 1/81	0.00085	0.00070	0.00051	0.00055
Apr. 8/81	0.00100	0.00089	0.00084	0.00084
Apr. 23/81	0.00122	0.00084	0.00061	0.00043
May 7/81	0.00124	0.00091	0.00054	0.00035
May 26/81	0.00129	0.00076	0.00045	0.00022
Oct. 8/81	0.00110	0.00082	0.00050	0.00029
Oct. 21/81	0.00102	0.00081	0.00050	0.00029
Nov. 12/81	0.00072	0.00050		
Nov. 26/81	0.00121	0.00102	0.00065	0.00034

The normalized roadbed deflections for each set of measurements have been plotted against depth below tie base in Exhibits No. 7 and 8. In spite of the previously mentioned scatter in data, the information does make sense.

On March 26 when the roadbed was frozen solid to a depth of roughly three feet below tie base, the frozen portion deflected as a solid mass, and there was no differential deflection between the 12-1/2 in. and 40 in. measurement points. As the roadbed underwent thawing during April, the granular deflections increased, deflections at top of subgrade increased, but deflection levels at 40 in. depth in subgrade actually decreased. By the time the roadbed was completely thawed, there were significant strain gradients in the subballast and subgrade.

The variations of deflection from winter through spring, summer and fall are plotted in Exhibit No. 9. It should be noted that there were no excessive roadbed deflections during the spring thaw, and that late April and May deflections were very similar to those recorded in October and November.

A plot of frost leaving the roadbed during April of 1982 is given in Exhibit No. 10 (note that Exhibit No. 9 was for roadbed deflections in 1981). While it would have been preferable to have frost tube readings on a daily basis throughout the thaw period, it appears that the frost layer finally melted somewhere within the subballast layer and not within the subgrade. This would have kept subgrade drainage paths open, and is consistent with the lack of excessive deflections during the 1981 spring thaw. There is probably a wealth of information available on Canadian frost depths and thawing time histories, but this matter has not been pursued because the spring thaw did not adversely affect the Joliette test site roadbed conditions.

2.4 Relationship between Wheel Loads and Tieplate Loads

A sample plot of wheel load vs. tieplate load is shown in Exhibit No. 11. There was scatter in data but a reasonable linear relationship existed. Ratios of average tieplate load to wheel load for the various measurement dates are shown in Table No. 3. Average tieplate loads ranged from 20% to 50% of the wheel loads. When work first started on this project, roadbed deflections were normalized with respect to tieplate loads because that is the reference British Rail had used in their roadbed analysis work. From CN measurements on the twelve Edmonton area test sites (reference Report in Bibliography), it was concluded that tieplate loads had tremendous variation and that wheel loads would provide a better reference because they were easier to categorize. One also had to recognize the fact that the roadbed had to support the wheel loads regardless of how they were distributed downward. It is felt that wheel load distributions remained relatively constant throughout the measurement period given in Table No. 3, but that variations in tieplate loading occurred. That is one variable that this study can do without, and it is for that reason in this Report that roadbed deflections have been normalized with respect to wheel loads rather than tieplate loads.

ROADBED AVERAGE COMPLIANCE VALUES
DEFLECTIONS NORMALIZED PER KIP OF WHEEL LOAD
SITE 18 MILE 118.2 JOLIETTE SUB., 1981

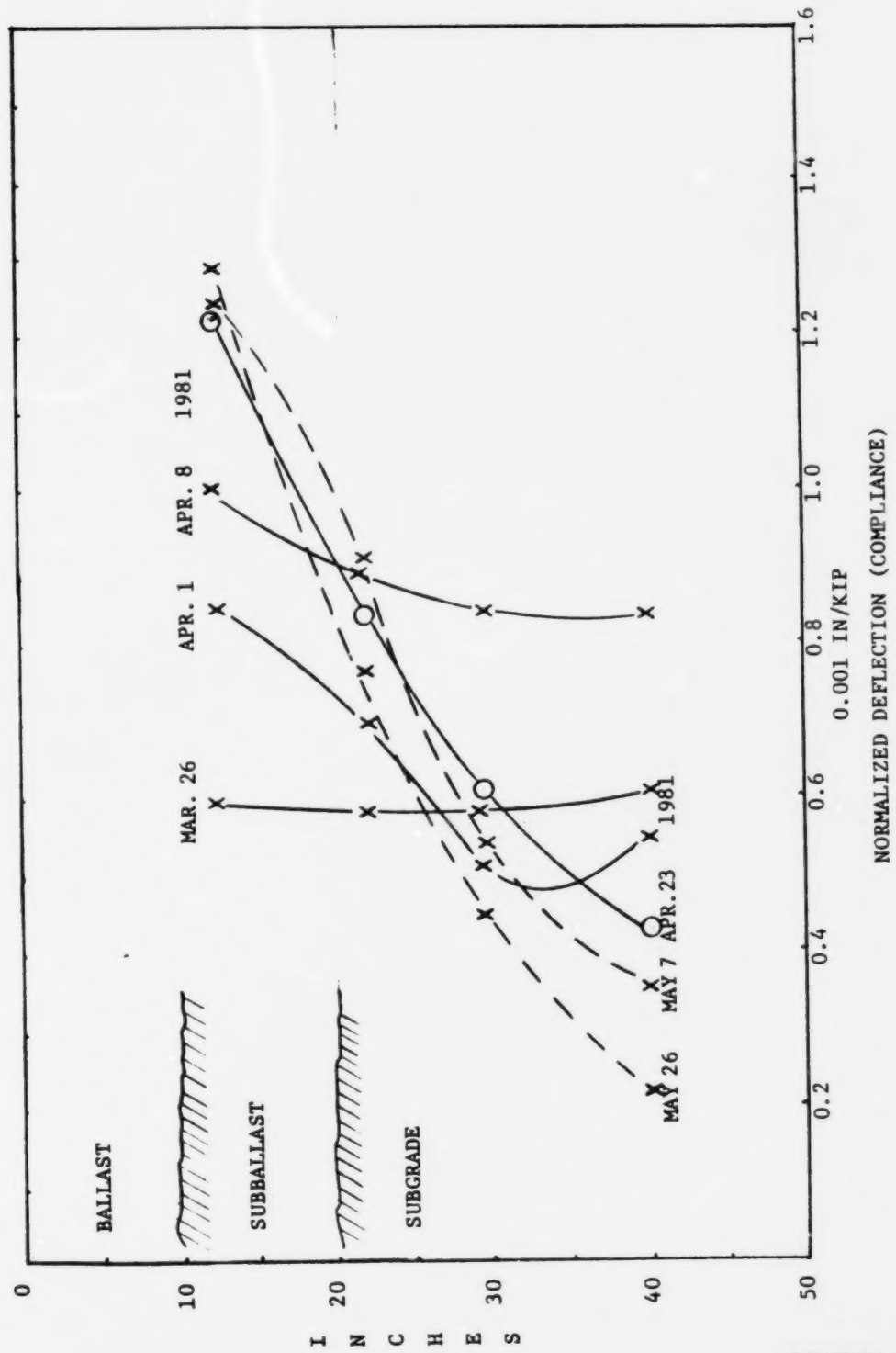


EXHIBIT NO. 7

D E P T H B E L O W B A S E O F T I E

ROADBED AVERAGE COMPLIANCE VALUES
DEFLECTIONS NORMALIZED PER KIP OF WHEEL LOAD
SITE 18 MILE 118.2 JOLIETTE SUB. 1981

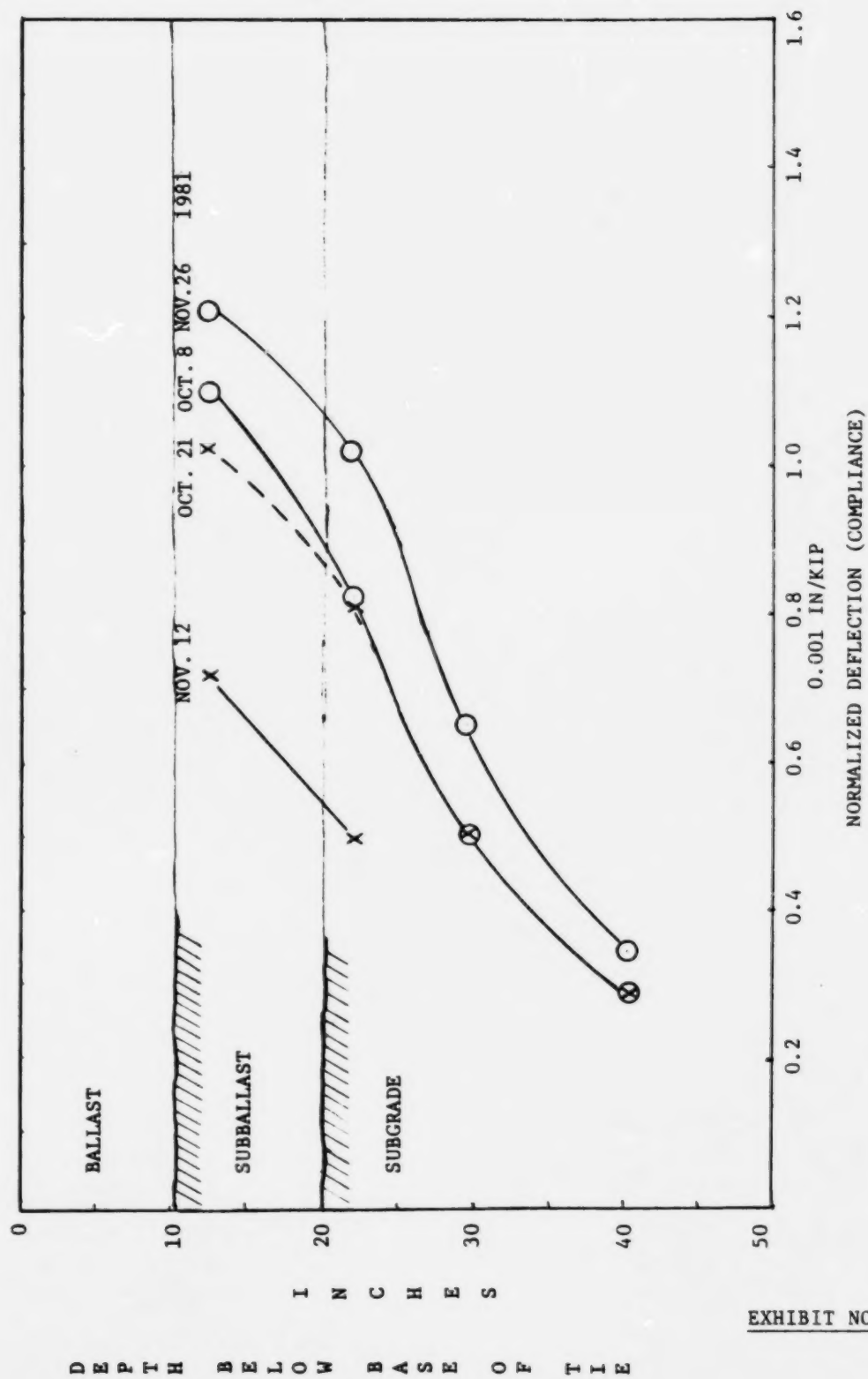
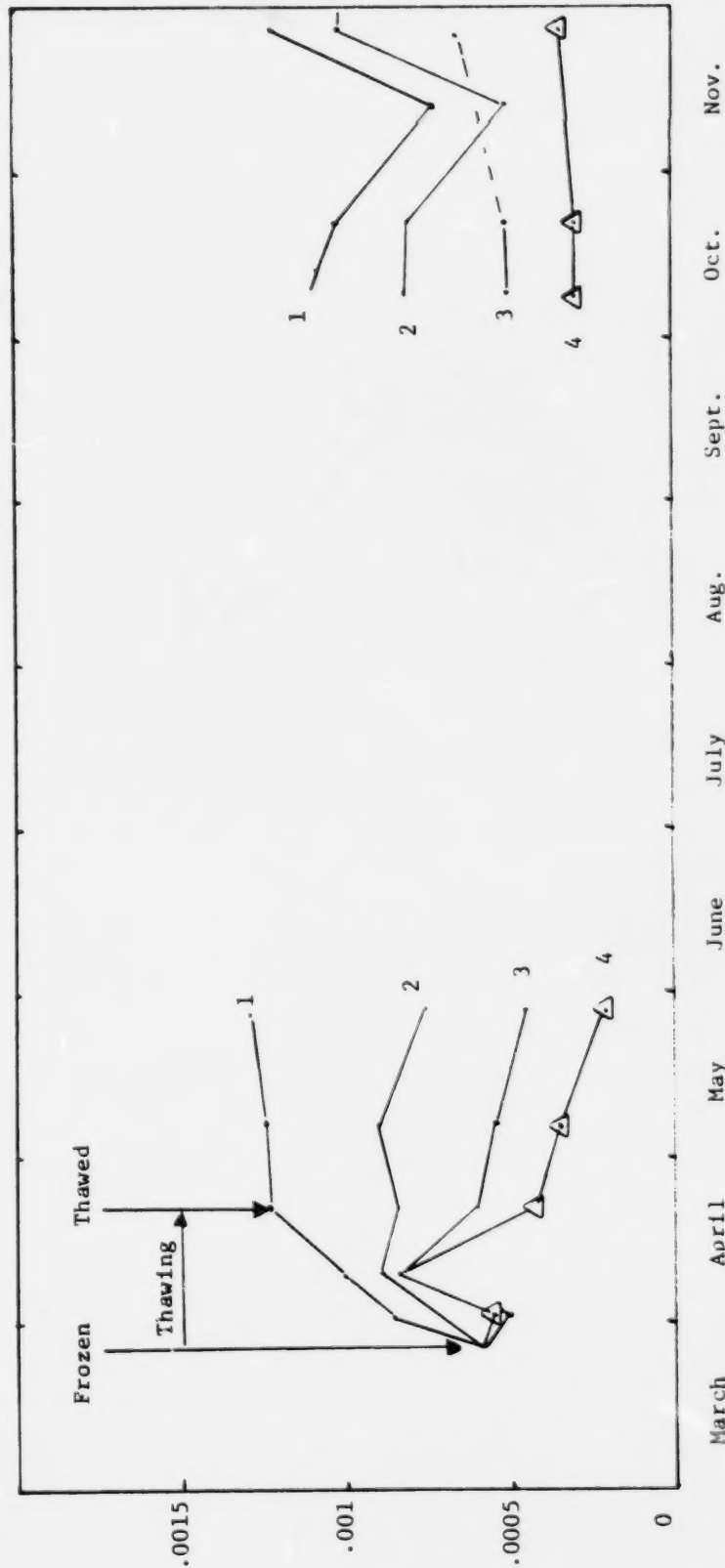


EXHIBIT NO. 8

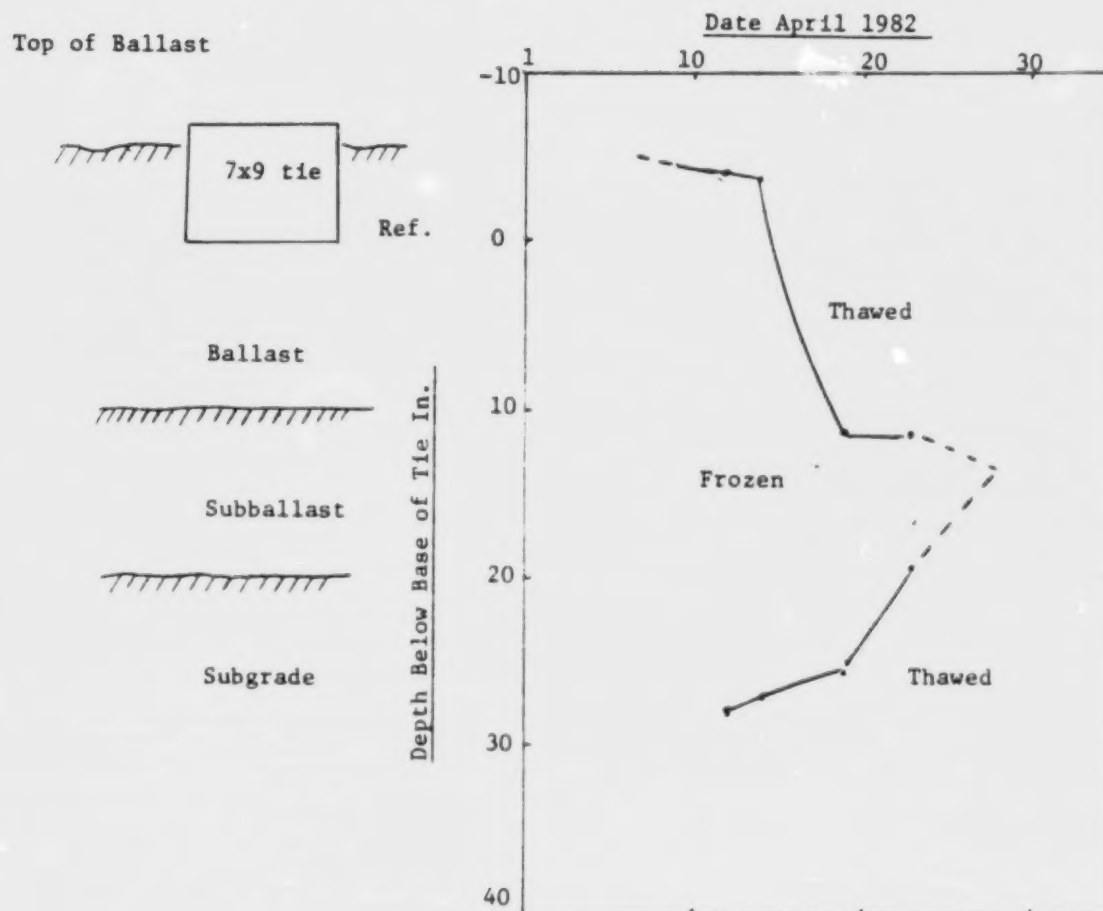
NORMALIZED ROADBED DEFLECTION

IN./KIP WHEEL LOAD

- At Depth 1, 12-1/2 in. below base of tie (within granular)
- At Depth 2, 22 in. below base of tie (in top of subgrade)
- At Depth 3, 29-1/2 in. below base of tie (within subgrade)
- At Depth 4, 40 in. below base of tie (within subgrade)



THAWING OF FROZEN ROADBED



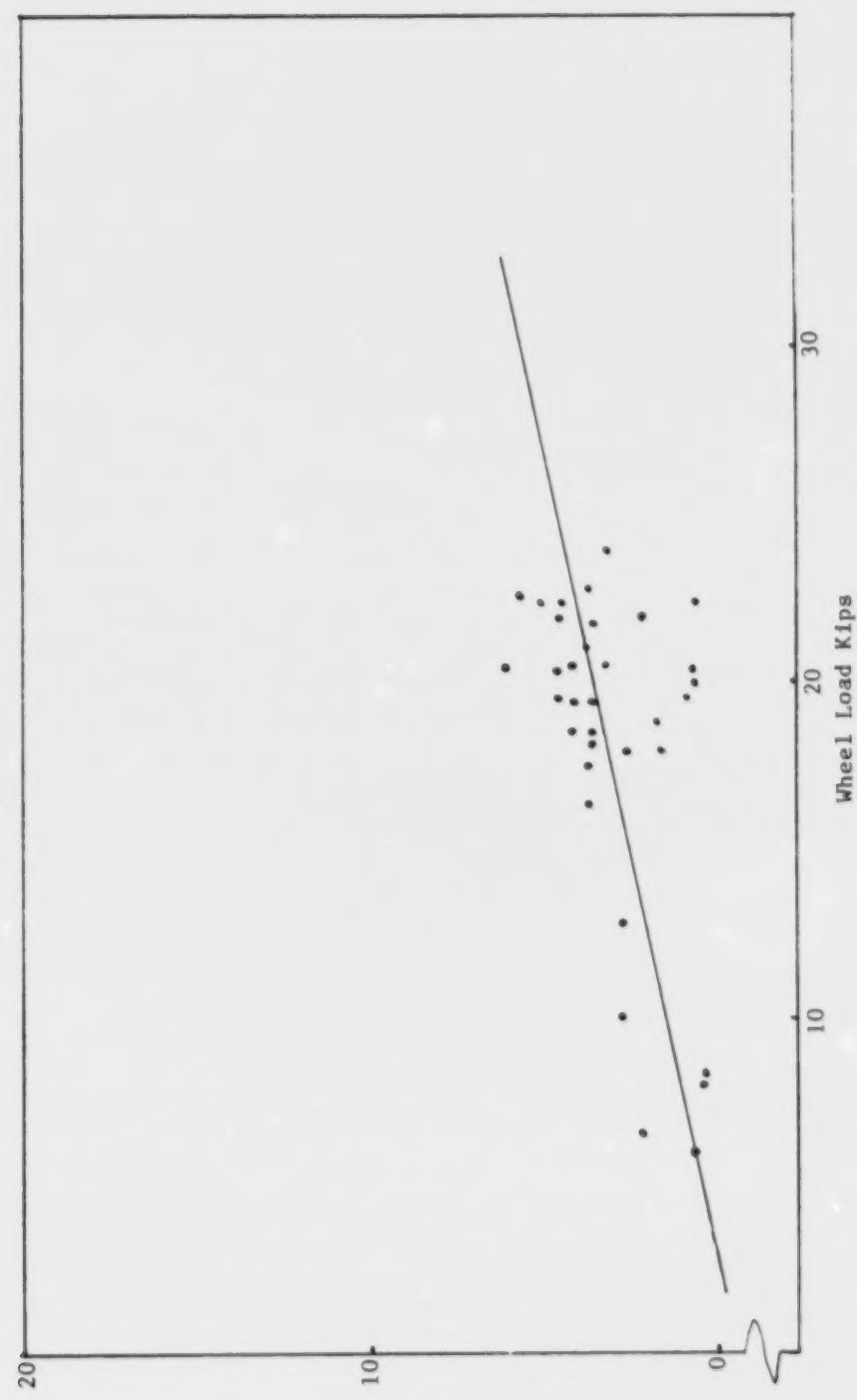
Depth of Frost Measured on West Frost Tube

Date 1982	Frost Line Measured From Top of Tie, In.	
	Top of Frost	Bottom of Frost
April 12	3	35
April 14	3-1/4	34
April 19	18-1/2	32-1/2
April 23	18-1/2	26-1/2
April 28	No Frost	No Frost

AVERAGE TIEPLATE LOAD vs. WHEEL LOAD

FREIGHT TRAIN RECORD NO. 20, MAY 26/81

SITE NO. 18 MILE 118.2 JOLIETTE SUB.



Average Tie Plate Load (Kips)

EXHIBIT NO. 11

TABLE NO. 3RATIO OF $\frac{\text{AVERAGE TIEPLATE LOAD}}{\text{WHEEL LOAD}}$ SITE NO. 18 MILE 118.2 JOLIETTE SUB.

(Average tieplate loading was plotted against wheel loading)

Date	Ratio $\frac{\text{Tieplate Load}}{\text{Wheel Load}}$
Mar. 26/81	0.28
Apr. 1/81	0.21
Apr. 8/81	0.27
Apr. 23/81	0.23
May 7/81	0.22
May 26/81	0.40
Oct. 8/81	0.38
Oct. 21/81	0.50
Nov. 12/81	0.49
Nov. 26/81	0.45

2.5 Roadbed Peak Absolute Deflections

While normalization of roadbed deflection with respect to wheel load provides the capability of being able to interpolate and extrapolate deflections throughout the entire wheel dynamic load distribution (from 2 to 50 kips nominal), the data analysis work is very tedious. A far quicker and easier approach is to simply analyze data from a few of the peak loadings. Roadbed peak absolute deflections with corresponding wheel loads have been listed in Table No. 4. These deflections have been plotted against depth of measurement in Exhibits No. 12 to 20 inclusive. (It should be noted that reference for depth in roadbed has been taken as base of tie throughout this Report.) The frozen roadbed deflected en masse as seen by the March 26 deflections of Exhibit No. 12. In Exhibit No. 13, there was obviously an error in reading at the 40 in. depth because the deflections at 30 in. depth were lower. While great care was taken in trying to accurately calibrate displacement transducers with 0.020 in. shims, it was difficult to maintain such precision on account of the track vibrational environment. In general, the deflection with depth curves were quite smooth, indicating the deflection gradients within the subballast and subgrade.

2.6 Soil Strain at Subgrade Surface

The purpose of the granular layer is to distribute loading to the subgrade such that the top of subgrade is not overstressed or overstrained, otherwise high elastic deflections and/or permanent settlements will occur. By measuring the slopes of the roadbed deflection with depth graphs, it has been possible to estimate the soil strains in the top surface layer of the subgrade. Soil strains derived from the normalized deflection curves for the Joliette test site have been listed in Table No. 5. It should be noted that the top of subgrade has been taken as 20 in. below base of tie for all these strain values. Strain levels given in percentage were determined per kip of wheel load from the normalized graphs. Strain levels for 25, 34.6, and 50 kip wheel loads were derived from simple linear extrapolation. These wheel loads were selected because 25 kips represents medium load traffic in the 20 to 30 kip range, 34.6 kips represents the average of all wheel loads exceeding 30 kips, and 50 kips represents the nominal maximum dynamic wheel load. It is felt that all wheel loads of less than 20 kips can be safely ignored on tracks carrying 100-ton cars. Comments on characterization of vertical wheel loads are given in Appendix D of this Report.

Subgrade surface maximum strain levels of 0.192% and 0.278% were determined for wheel loads of 34.6 and 50 kips. These strain levels appear to be low in comparison to the threshold level of 0.65% quoted by British Rail for London clays.

A plot of subgrade surface strain vs. wheel load for the various measurement dates is given in Exhibit No. 21. The lines are

TABLE NO. 4

ROADBED PEAK ABSOLUTE DEFLECTIONS vs. WHEEL LOADS

SITE NO. 18 MILE 118.2 JOLIETTE SUB.

DATE	WHEEL LOAD	SUBGRADE 1	SUBGRADE 2	SUBGRADE 3	SUBGRADE 4
MAR. 26/81	29.7	0.017	0.017	0.015	0.016
	27.2	0.018	0.018	0.017	0.017
	25.9	0.022	0.021	0.019	0.019
	17.3	0.008	0.008	0.007	0.007
	6.5	0.007	0.007	0.006	0.006
APR. 1/81	40.9	0.025	0.019	0.015	0.017
	36.2	0.027	0.023	0.019	0.021
	33.9	0.024	0.007	0.012	0.014
	19.1	0.010	0.001	0.005	0.006
	7.6	0.005	0.004	0.003	0.003
APR. 8/81	33.6	0.022	0.015	0.011	0.014
	32.8	0.019	0.014	0.010	0.012
	33.2	0.028	0.028	0.026	0.026
	20.0	0.017	0.016	0.014	0.014
	8.0	0.003	0.003	0.002	0.002
APR. 23/81	31.0	0.031	0.018	0.011	0.005
	30.4	0.032	0.019	0.010	0.004
	23.7	0.029	0.020	0.012	0.005
	20.0	0.020	0.009	0.006	0.001
	9.3	0.007	0.005	0.003	0.001
MAY 7/81	27.4	0.032	0.021	0.015	0.011
	26.0	0.032	0.021	0.013	0.007
	23.5	0.028	0.017	0.010	0.004
	15.4	0.014	0.010	0.007	0.003
	9.4	0.007	0.004	0.003	0.001
MAY 26/81	32.7	0.032	0.014	0.008	0.005
	33.4	0.042	0.025	0.016	0.005
	32.5	0.037	0.020	0.015	0.013
	20.6	0.023	0.013	0.008	0.001
	9.1	0.009	0.006	0.005	0.002
OCT. 8/81	37.8	0.034	0.025	0.017	0.007
	36.7	0.044	0.031	0.022	0.013
	36.7	0.036	0.025	0.017	0.009
	19.4	0.024	0.018	0.014	0.006
	8.7	0.014	0.008	0.007	0.001
OCT. 21/81	34.3	0.030	0.024	0.016	0.010
	34.1	0.032	0.024	0.016	0.009
	35.7	0.037	0.028	0.019	0.012
	20.0	0.024	0.017	0.012	0.005
	7.8	0.006	0.004	0.003	0.001
NOV. 12/81	28.7	0.015	0.010		
	25.6	0.016	0.011		
	21.3	0.015	0.011		
	15.0	0.009	0.008		
	6.4	0.002	0.001		
NOV. 26/81	27.8	0.025	0.022	0.014	0.007
	29.3	0.027	0.024	0.016	0.008
	25.7	0.027	0.023	0.015	0.007
	20.6	0.020	0.016	0.010	0.004
	9.8	0.008	0.009	0.006	0.002

ROADBED PEAK ABSOLUTE DEFLECTIONS
SITE 18 MILE 118.2 JOLIETTE SUB. MAR. 26, 1981

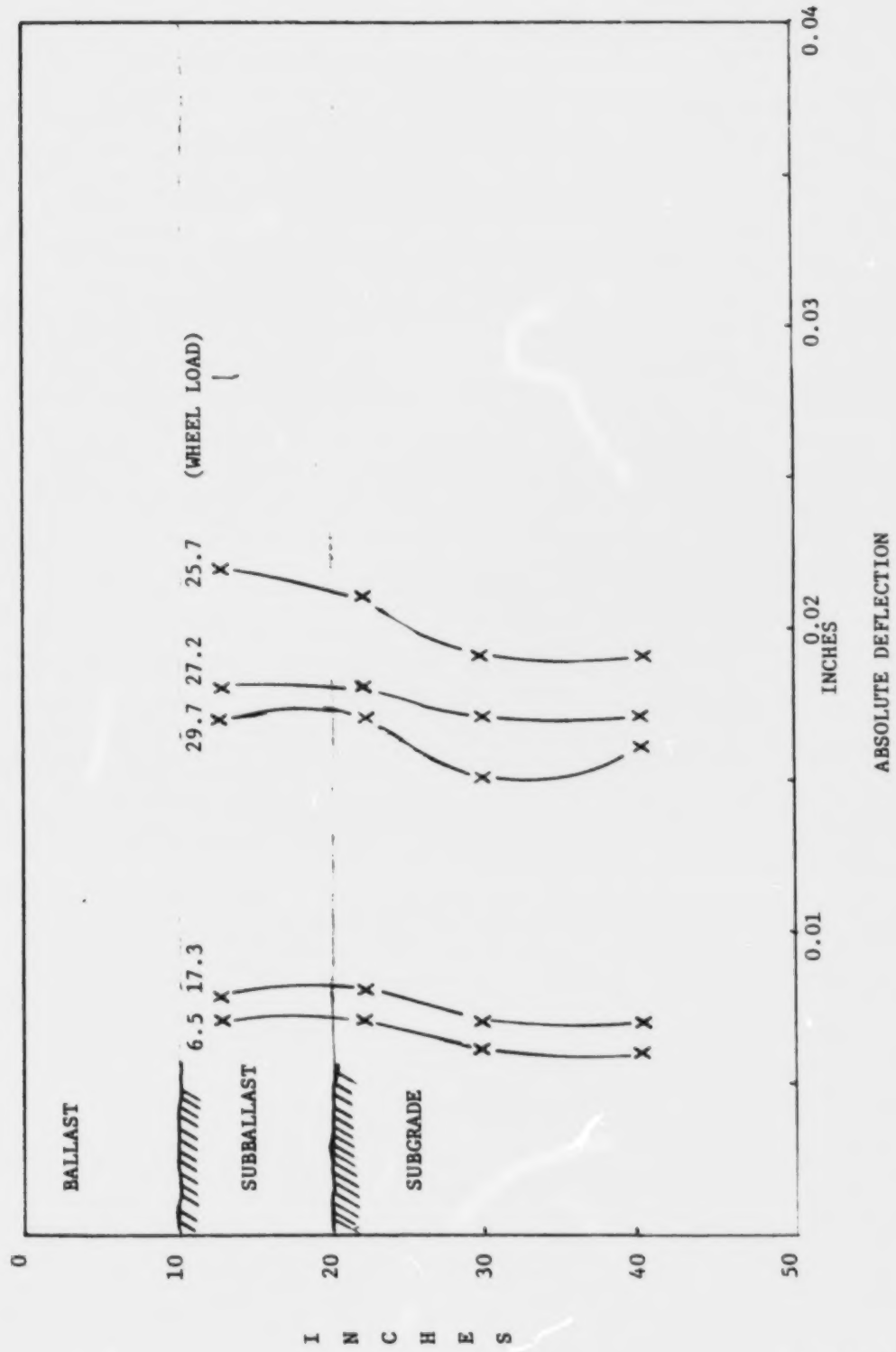


EXHIBIT NO. 12

DEPTH BELOW BASE OF TIE

ROADBED PEAK ABSOLUTE DEFLECTIONS
SITE 18 MILE 118.2 JOLIETTE SUB. APR. 1, 1981

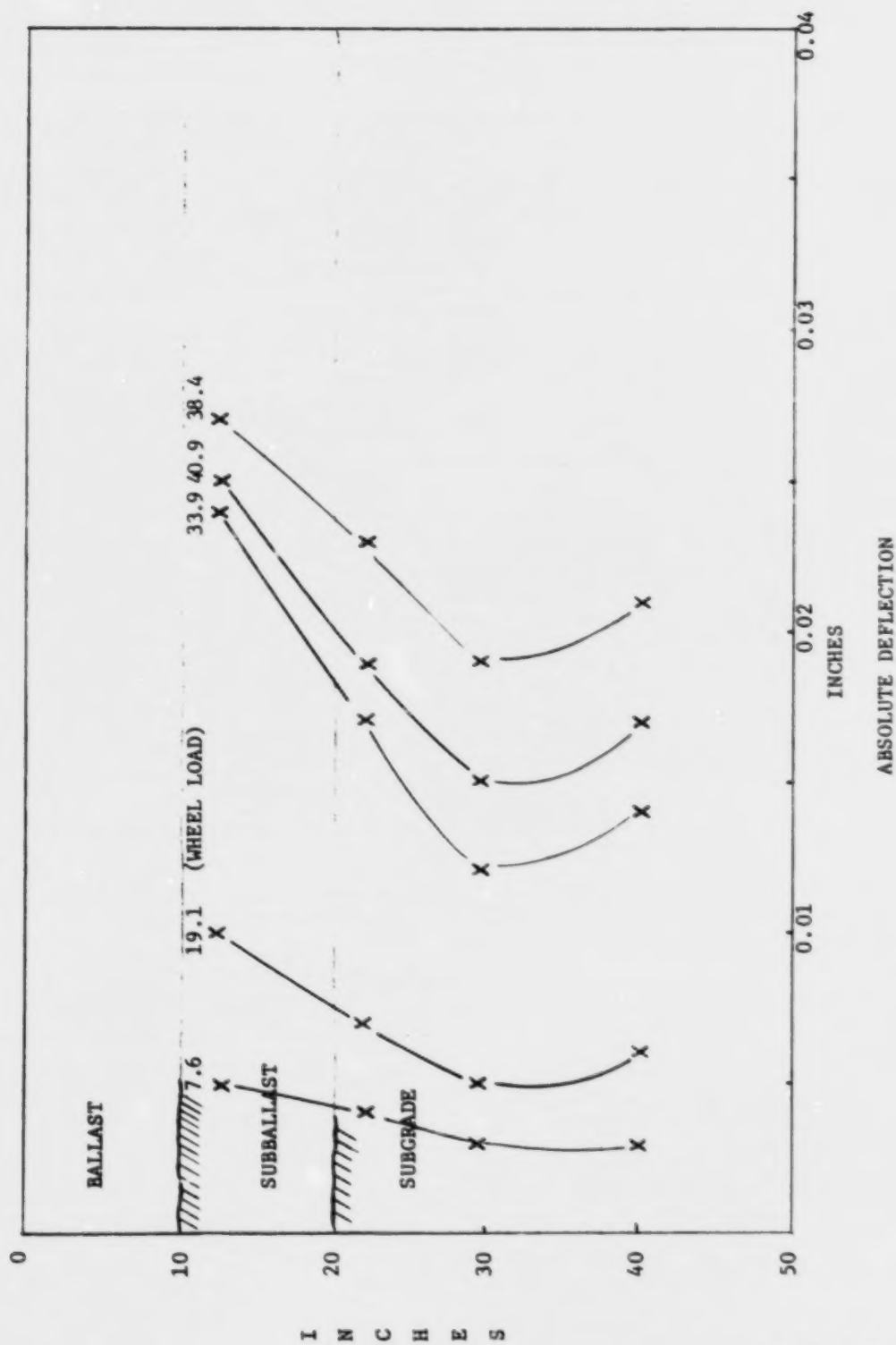


EXHIBIT NO. 13

D E P T H B E L L O W B A S E O F T I E

ROADBED PEAK ABSOLUTE DEFLECTIONS
SITE 18 MILE 118.2 JOLIETTE SUB. APR. 8, 1981

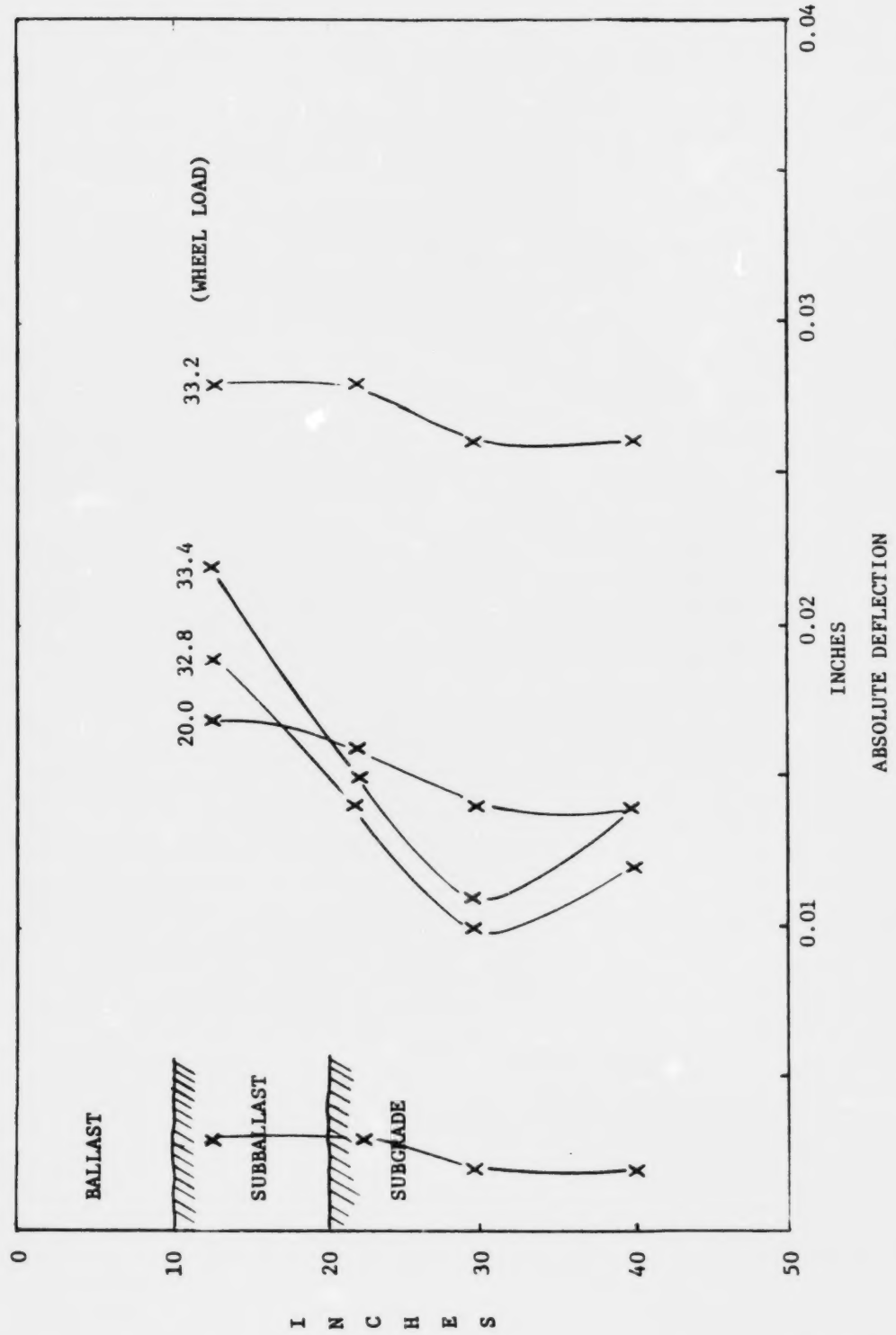


EXHIBIT NO. 14

DEPTH BELOW BASE OF TIE

ROADBED PEAK ABSOLUTE DEFLECTIONS
SITE 18 MILE 118.2 JOLIETTE SUB. APR. 23, 1981

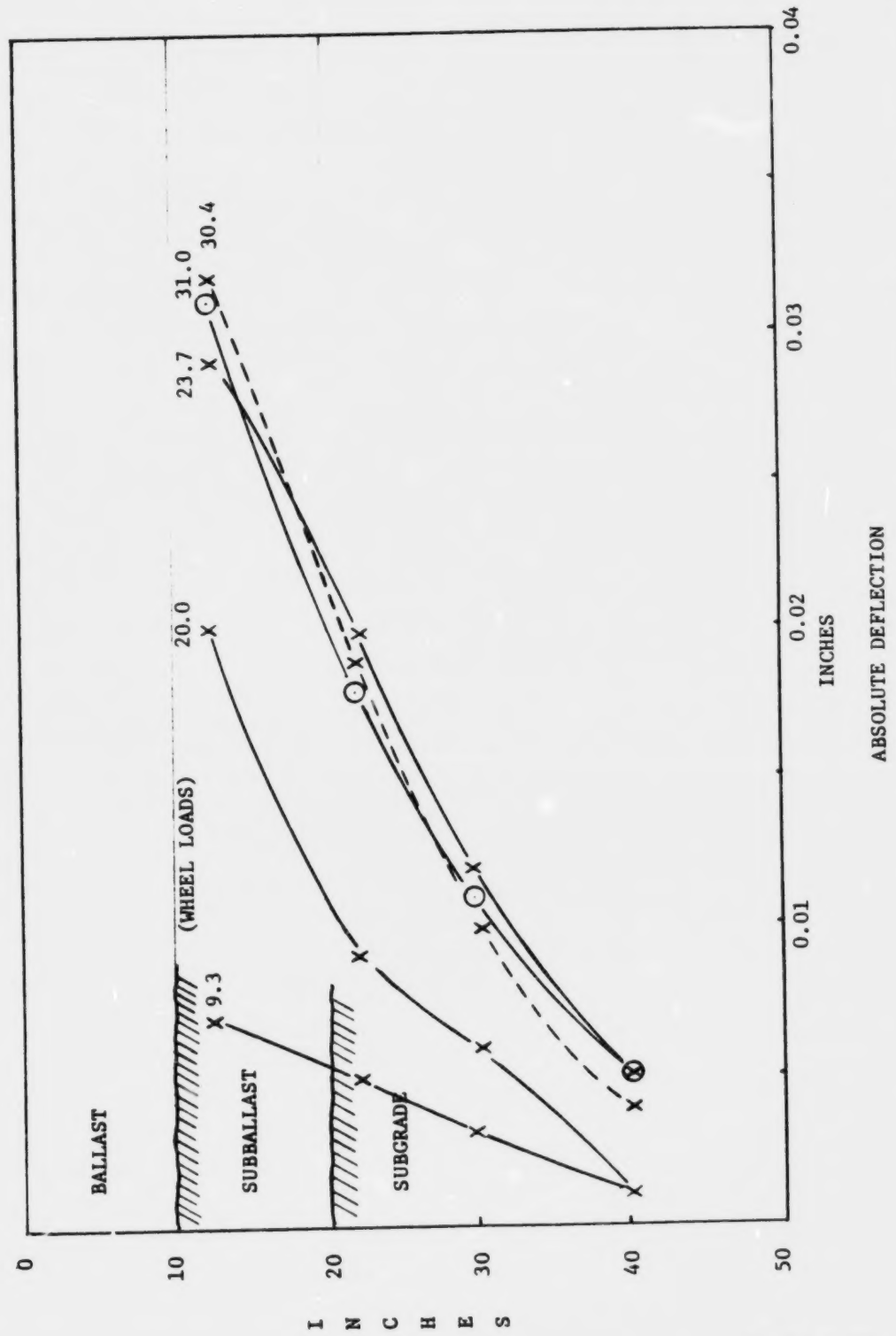
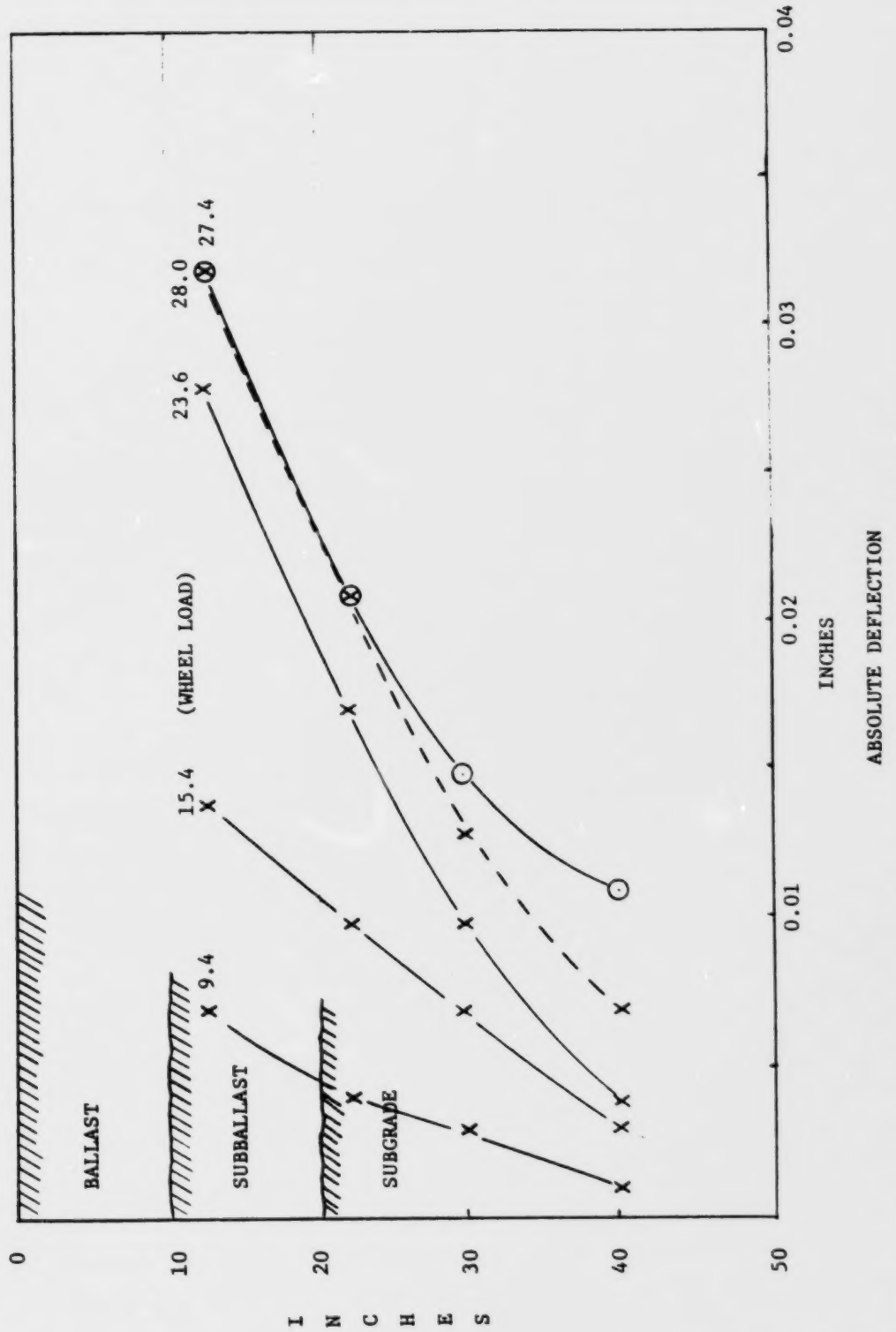


EXHIBIT NO. 15

D E P T H B E L O W B A S E O F T I E

ROADBED PEAK ABSOLUTE DEFLECTIONS
SITE 18 MILE 118.2 JOLIETTE SUB. MAY 7, 1981



ROADBED PEAK ABSOLUTE DEFLECTIONS
SITE 18 MILE 118.2 JOLIETTE SUB. MAY 26, 1981

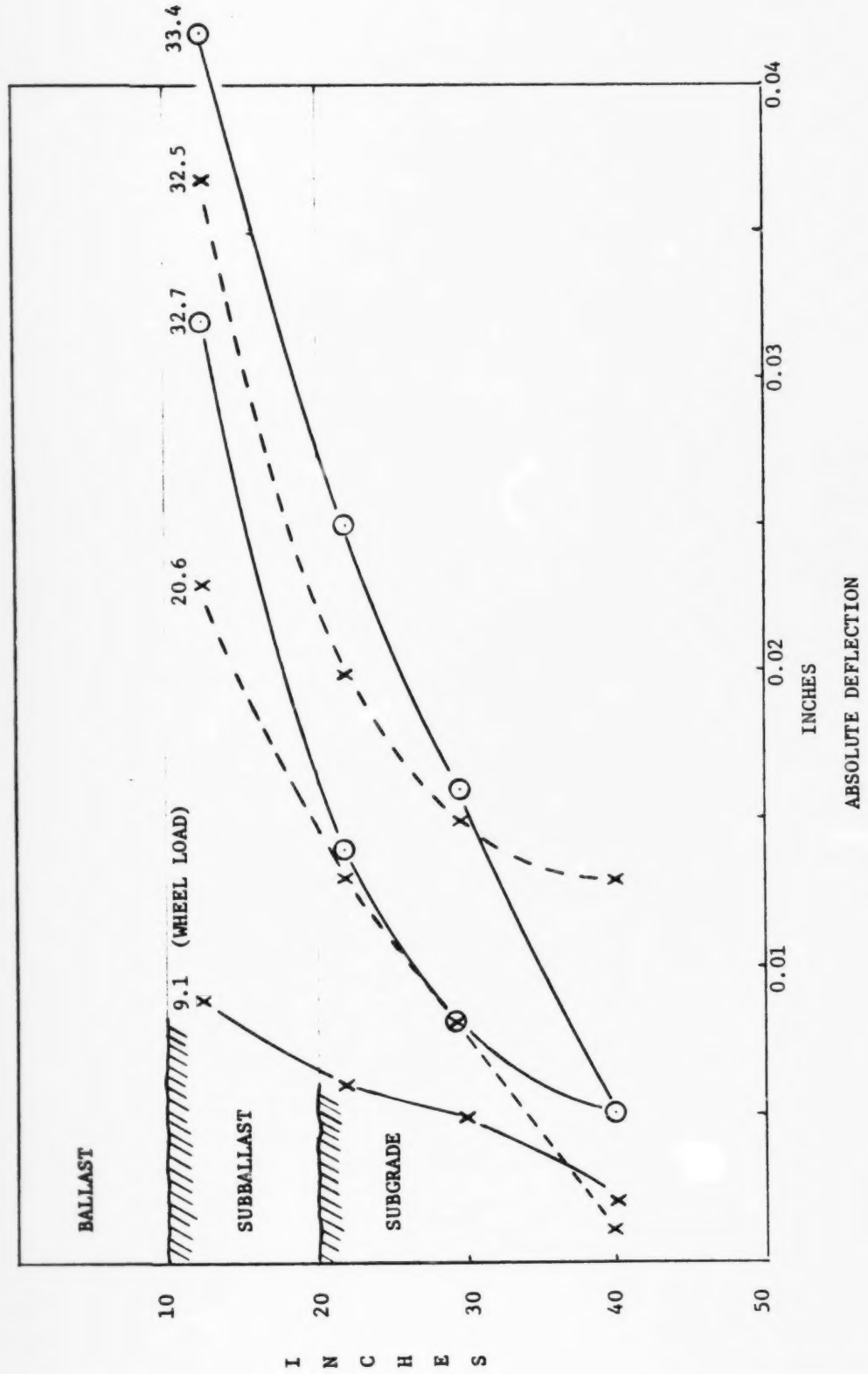


EXHIBIT NO. 17

DEPTH BELOW BASE OF TIE

ROADBED PEAK ABSOLUTE DEFLECTIONS
SITE 18 MILE 118.2 JOLIETTE SUB. OCT. 8, 1981

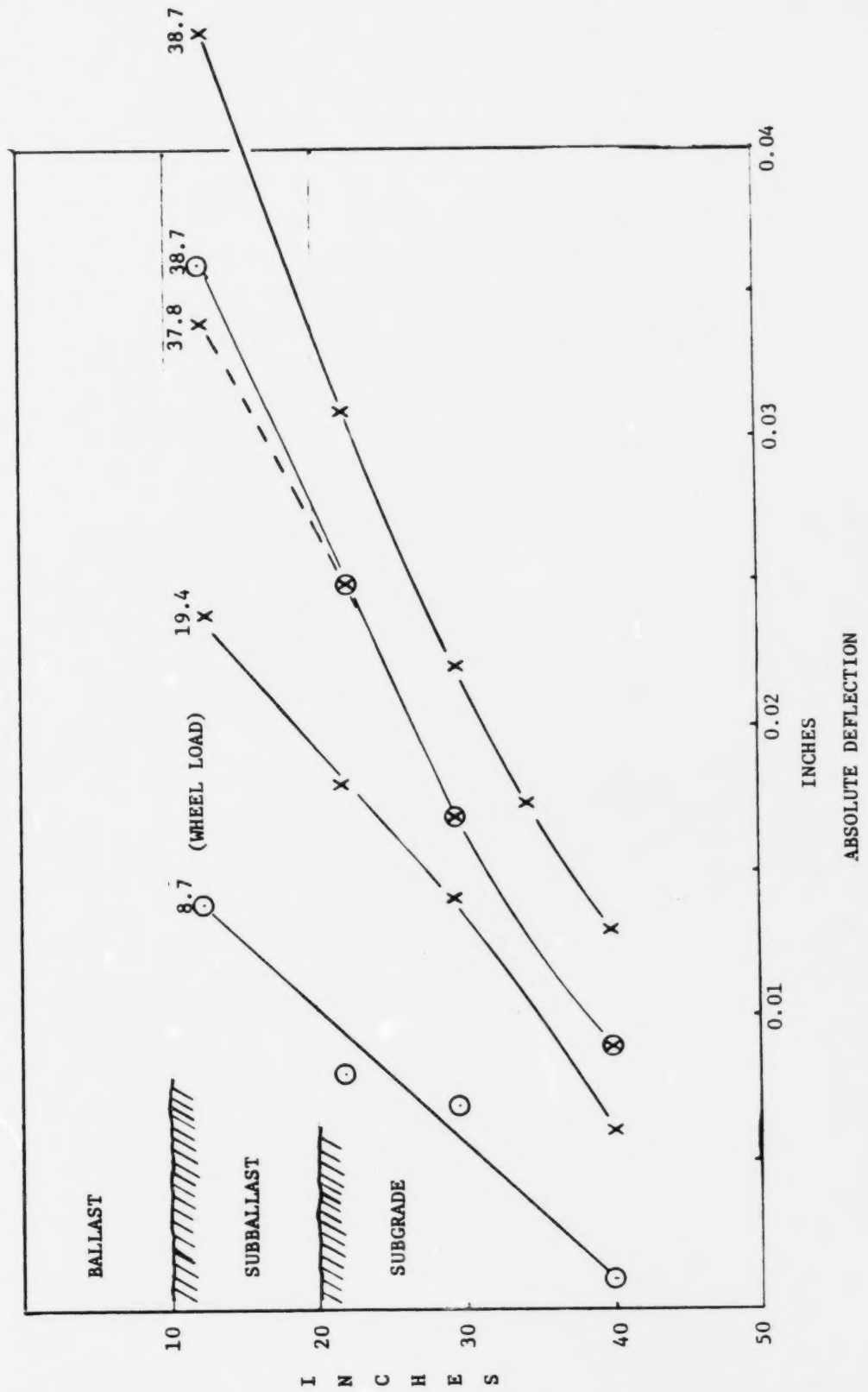


EXHIBIT NO. 18

DEPTH BELOW BASE OF TIE

ROADBED PEAK ABSOLUTE DEFLECTIONS
 SITE 18 MILE 118.2 JOLIETTE SUB. OCT. 21, 1981

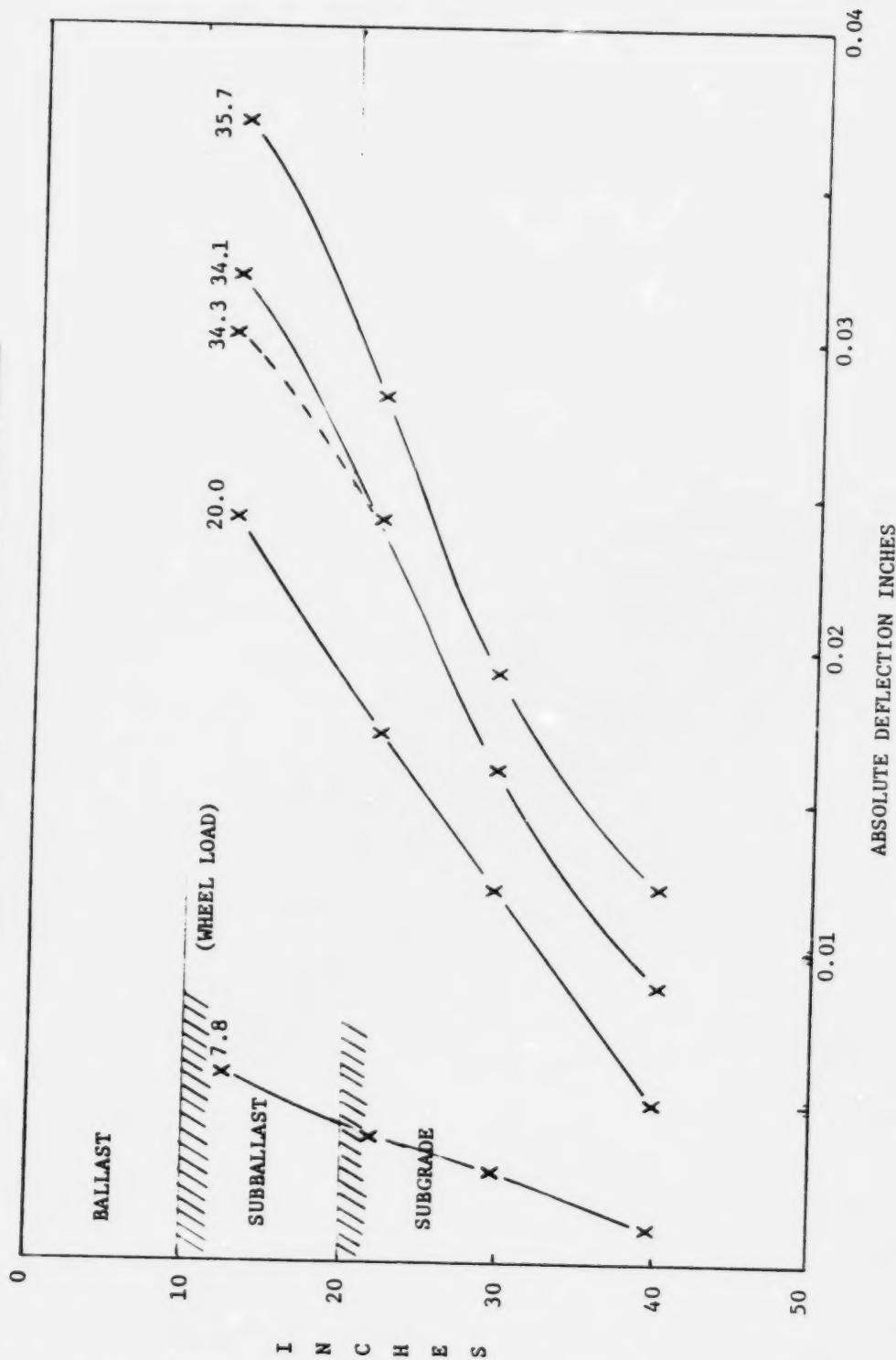


EXHIBIT NO.19

DEPTH BELOW BASE OF TIE

ROADBED PEAK ABSOLUTE DEFLECTIONS
SITE 18 MILE 118.2 JOLIETTE SUB. NOV. 26, 1981

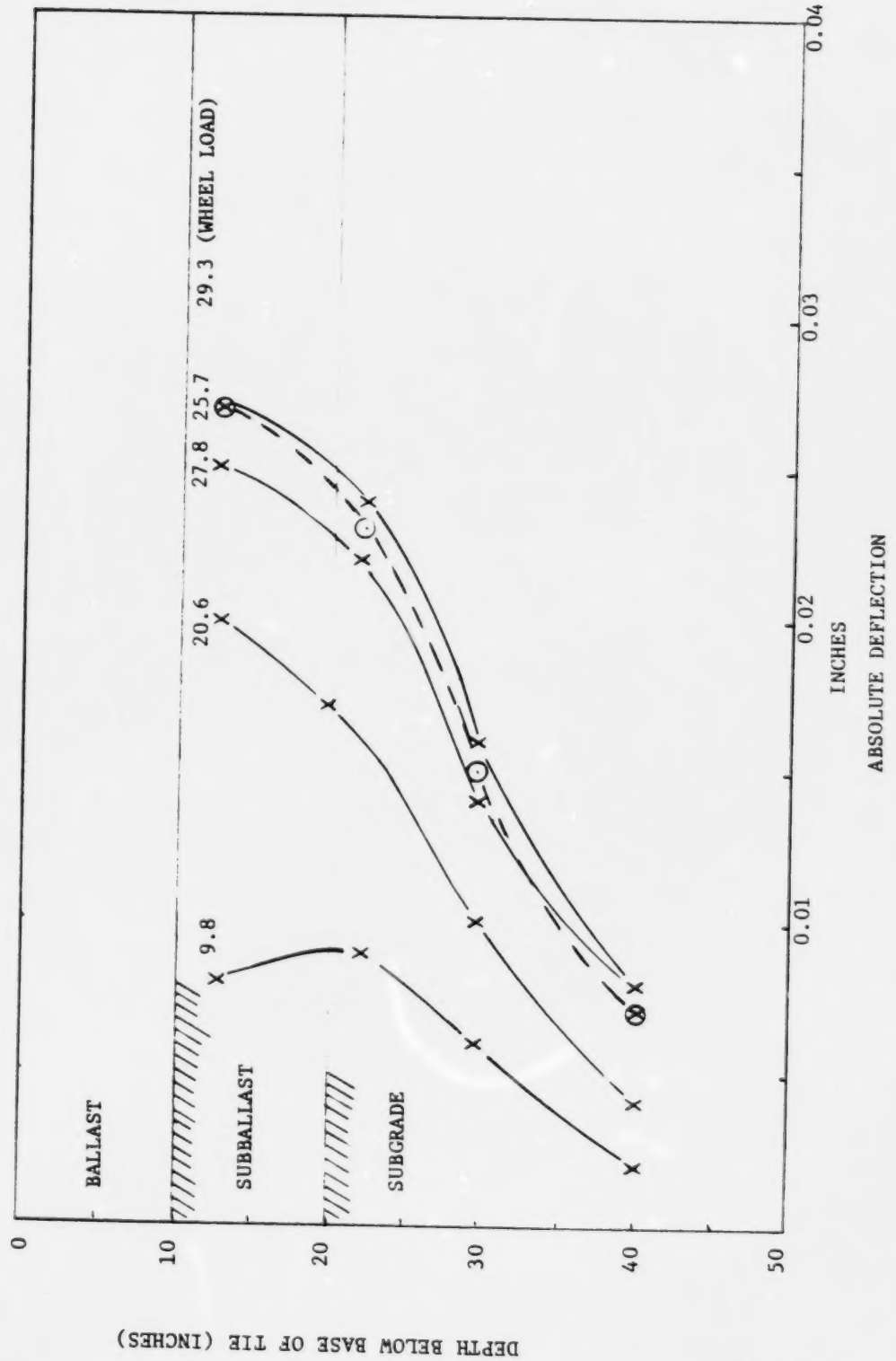


TABLE NO. 5
SOIL STRAINS AT SUBGRADE SURFACE
SITE NO. 18 MILE 118.2 JOLIETTE SUB.

(Strains derived from normalized deflection curves.)

Date	Z Soil Strain at Subgrade Surface per Wheel Load				
	1 Kip	25 Kip	34.6 Kip	50 Kip	Comment
Mar. 26/81	-0	-0	-0	-0	Frozen
Apr. 1/81	.00203	.051	.070	.102	Frozen
Apr. 8/81	.00107	.027	.037	.054	Frozen
Apr. 23/81	.00350	.088	.121	.176	Thawing
May 7/81	.00465	.116	.161	.232	Thawed
May 26/81	.00554	.139	.192	.278	"
Oct. 8/81	.00388	.097	.134	.194	"
Oct. 21/81	.00299	.075	.103	.150	"
Nov. 12/81	.00228	.057	.079	.114	"
Nov. 26/81	.00324	.081	.112	.162	"

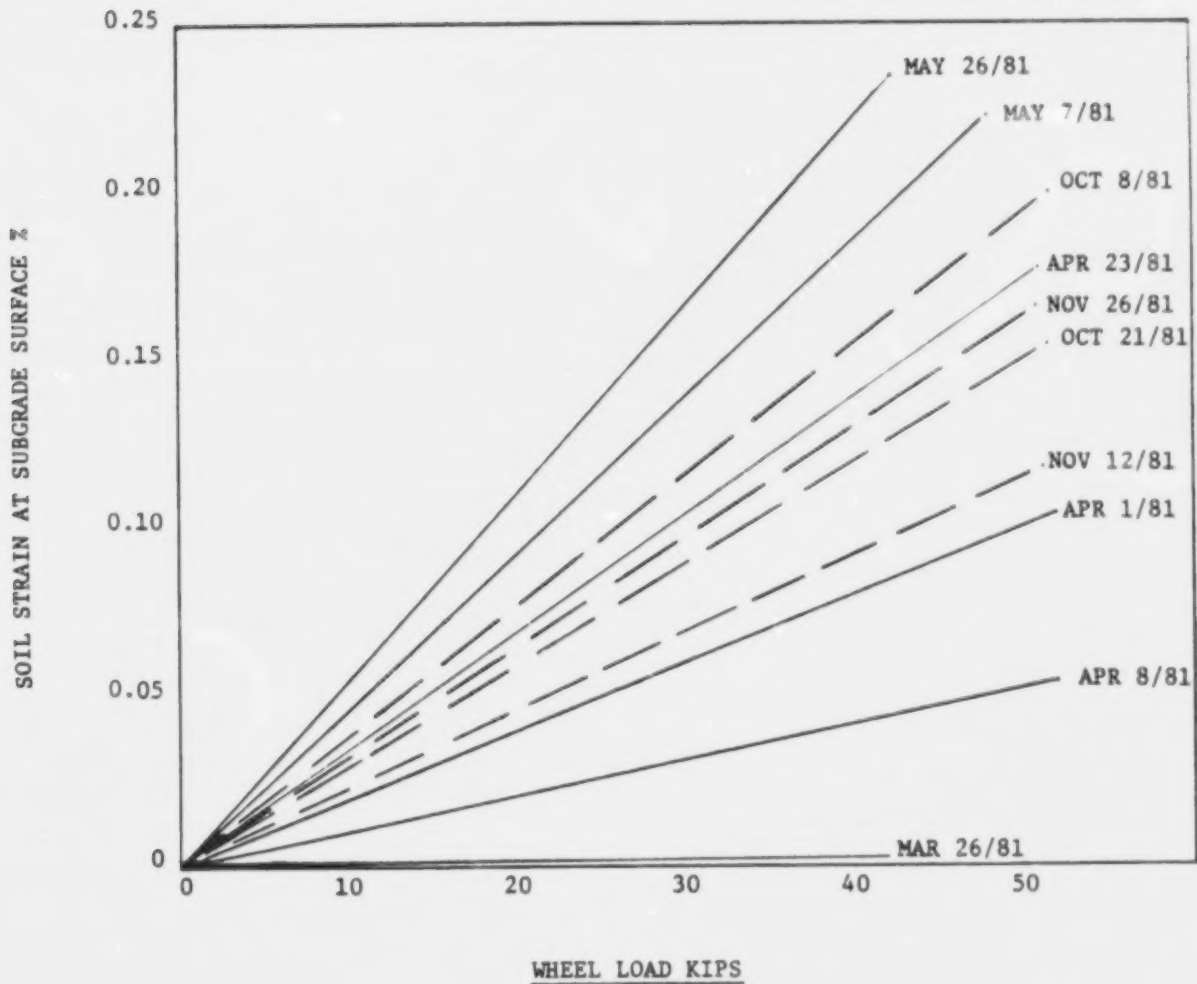
Note: Wheel load of 25 kips represents medium load traffic.

Wheel load of 34.6 kips represents average of heavy load traffic (for all wheels exceeding 30 kips).

Wheel load of 50 kips represents nominal maximum loading (generated by defective wheels, occurrence ~0.007% of total wheels).

SOIL STRAIN AT SUBGRADE SURFACE VS WHEEL LOAD
STRAINS DERIVED FROM NORMALIZED DEFLECTION CURVES
SITE NO. 18 MILE 118.2 JOLIETTE SUB.

Note: Linear plots because best-fit straight lines
were drawn through scatter of points on
roadbed deflection/wheel load graphs.



straight because the previous normalization process eliminated the shotgun effect from the data (the scatter should not be forgotten however).

From virtually zero strain on March 26 when the roadbed was frozen solid, strain levels increased until they reached a peak on May 26. This may be associated with the wet subgrade following spring thaw.

Soil strains determined from peak roadbed deflections have been listed in Table No. 6 and plotted against corresponding wheel loads in Exhibit No. 22. The graph is essentially the same as that of Exhibit No. 21, but only the peak strain envelope has been drawn. It took roughly one-twentieth of the effort to generate Exhibit No. 22 compared to Exhibit No. 21. It is easy to conclude which analysis technique will be used if future measurements are taken. Roadbeds must be constructed to withstand peak loadings.

If one eliminated the frozen and thawing roadbed data points from Exhibit No. 22, there remains a fair scatter in data. Could anyone develop a theory which would predict or encompass such scatter, or should we simply concentrate on field measurement and let the data speak for itself?

For direct comparison purposes, data measured in August of 1980 has been extracted from a previous Report⁽²⁾ and presented here as Table No. 7 and Exhibits No. 23 and 24. The data correlates in general with the 1981 data.

It is clear that the track and roadbed structure did not behave in a well-defined or consistent manner. While shotgun blasts of data can be linearized, normalized or fitted with smooth theoretical curves, the scatter remains a fact of life. It is believed that such behaviour should be defined through statistical measurements, and until such statistics are developed, we will not be able to validate any roadbed design theory. It is for this reason that this Report is devoid of any theory - theory must be developed to suit the behaviour of the roadbed because the roadbed will not alter its behaviour to suit any theory. We must put the horse before the cart.

TABLE NO. 6

SOIL STRAIN AT SUBGRADE SURFACE

STRAINS DETERMINED FROM PEAK ROADBED DEFLECTIONS

SITE NO. 18 MILE 118.2 JOLIETTE SUB.

Wheel Load Kips	Strain ϵ %	Wheel Load Kips	Strain ϵ %	Wheel Load Kips	Strain ϵ %
Mar. 26/81 (Frozen)		Apr. 23/81 (Thawing)		Oct. 8/81	
6.5	0	9.3	.023	8.7	.046
17.3	0	20.0	-.063	19.4	.060
25.7	.01	23.7	.093	37.8	.120
27.2	0	30.4	.113	38.7	.120
29.7	0	31.0	.114	38.7	.122
Apr. 1/81 (Frozen)		May 7/81 (Thawed)		Oct. 21/81	
7.6	.01	9.4	.021	7.8	.020
19.1	.03	15.4	.043	20.0	.070
33.9	.072	23.6	.119	34.1	.105
38.4	.045	27.4	.119	34.3	.087
40.9	.060	28.0	.119	35.7	.120
Apr. 8/81 (Frozen)		May 26/81		Nov. 26/81	
8.0	.013	9.1	.021	9.8	.038
20.0	.026	20.6	.079	20.6	.076
32.8	.055	32.5	.112	25.7	.102
33.2	.028	32.7	.113	27.8	.105
33.4	.070	33.4	.142	29.3	.107

SOIL STRAIN AT SUBGRADE SURFACE VS WHEEL LOAD

STRAINS DETERMINED FROM PEAK DEFLECTIONS

SITE NO. 18 MILE 118.2 JOLIETTE SUB.

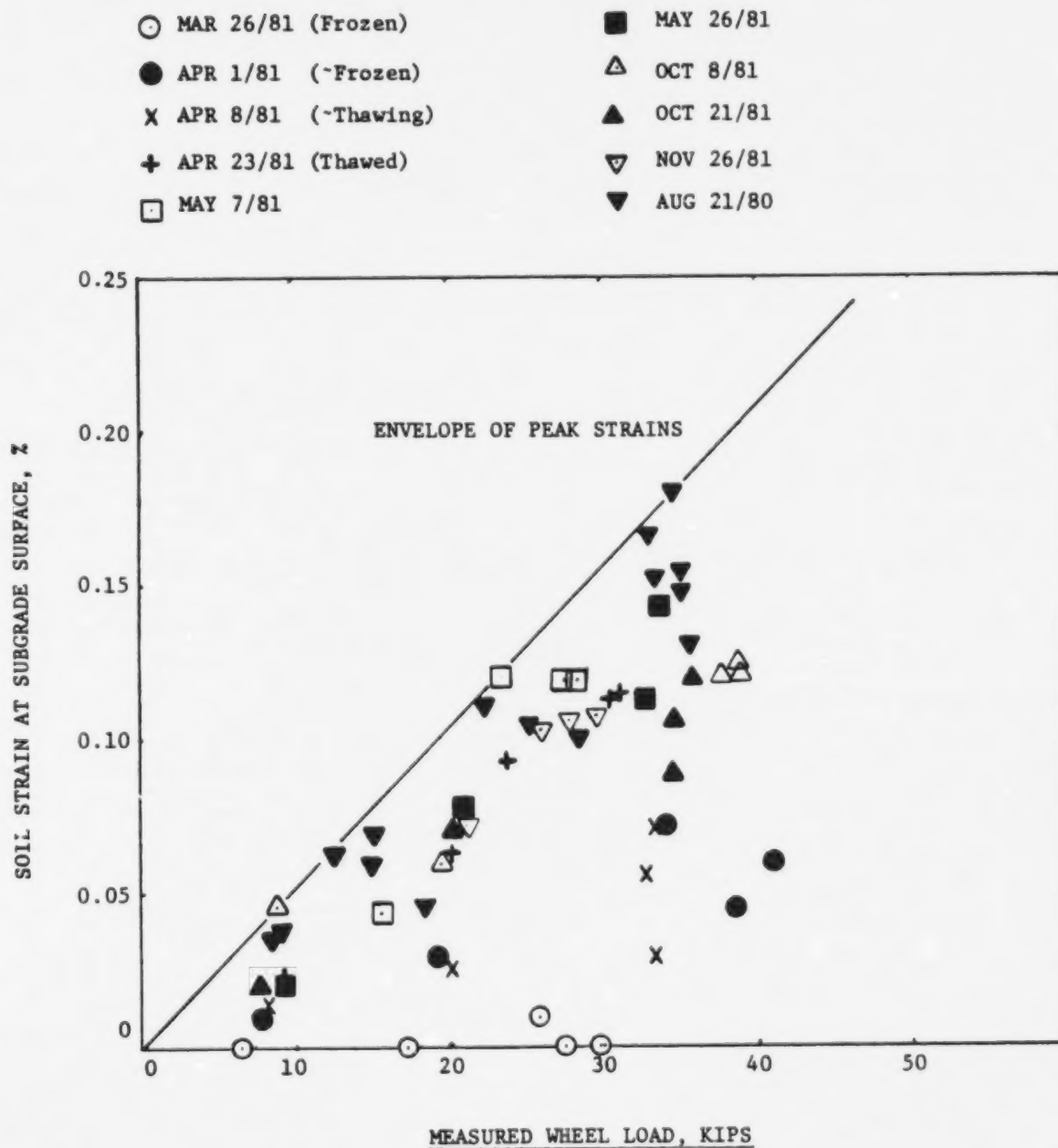


TABLE NO. 7

PEAK ABSOLUTE LOADS AND ROADBED DEFLECTIONSSOIL STRAINS AT SUBGRADE SURFACE

SITE NO. 18 MILE 118.2 JOLIETTE SUB. AUG. 21/80

(1) Determined from Peak Deflections; (2) Determined from Normalized Deflections (Compliance).

Rdg. No.	Rec. No.	Loads, Kips				Peak Absolute Roadbed Deflections, In.						Soil Strain ϵ @ Subgrade Surface (20")		
		Left Tie	Right Tie	Avg. Tie	Wheel	Left Tie	Right Tie	Avg. Tie	Pt. (1) 12-1/2"	Pt. (2) 22"	Pt. (3) 29-1/2"	Pt. (4) 40"	Peak (1)	Compl. (2)
a	2	25.0	15.0	18.0	28.4	.071	.254	.162	.035	.022	.016	.008	.100	.144
b	2	14.0	11.0	12.0	15.0	.048	.251	.145	.021	.016	.011	.005	.059	.096
c	3	28	19.5	20.8	35.8	.080	.279	.174	.042	.029	.021	.010	.130	.166
d	4	27.5	17.7	21.0	33	.079	.248	.138	.044	.029	.018	.009	.166	.168
e	4	24.5	16.5	20.6	27.5	.075	.248	.136	.029	.028	.017	.009	-	.165
f	4	34.4	21.8	25.8	34.5	.091	.254	.146	.051	.034	.020	.009	.181	.206
g	2	12.0	9.0	10.0	18.5	.038	.235	.135	.016	.012	.008	.0035	.045	.080
h	3	11.0	10.0	10.0	12.5	.050	.245	.145	.022	.0155	.0118	.0055	.062	.080
i	3	13.0	9.5	10.5	19.5	.048	.245	.148	.0225	.016	.012	.006	.062	.084
j	4	32	19.0	22	33.5	.085	.240	.155	.0475	.033	.023	.013	.151	.176
k	4	14	10.0	12	15	.045	.200	.115	.024	.0165	.012	.0075	.069	.096
l	4	14	9.5	12	15	.040	.200	.1175	.024	.0165	.0125	.0075	.069	.096
m	4	6	6	6	9	.030	.250	.125	.013	.009	.007	.004	.039	.048
n	4	23	16	20	27.5	.075	.290	.175	.045	.032	.024	.0135	.125	.160
o	4	21	13	17	25	.060	.310	.180	.036	.025	.018	.010	.104	.136
p	4	22	13	17.5	22.5	.060	.220	.140	.038	.0265	.019	.0115	.111	.140
q	4	5	6	5	8.5	.036	.170	.100	.011	.008	.007	.004	.035	.040
r	4	31	21.5	25.5	35	.085	.335	.205	.053	.035	.0255	.015	.147	.204
s	4	33	21	25	35	.085	.335	.205	.053	.037	.027	.016	.147	.20
t	4	29	19	25.5	36	.085	.340	.210	.054	.038	.028	.017	.153	.204

PEAK ABSOLUTE LOADS AND ROADBED DEFL'N

SITE NO. 18 MILE 118.2 JOLIETTE SUB. AUG. 21/80

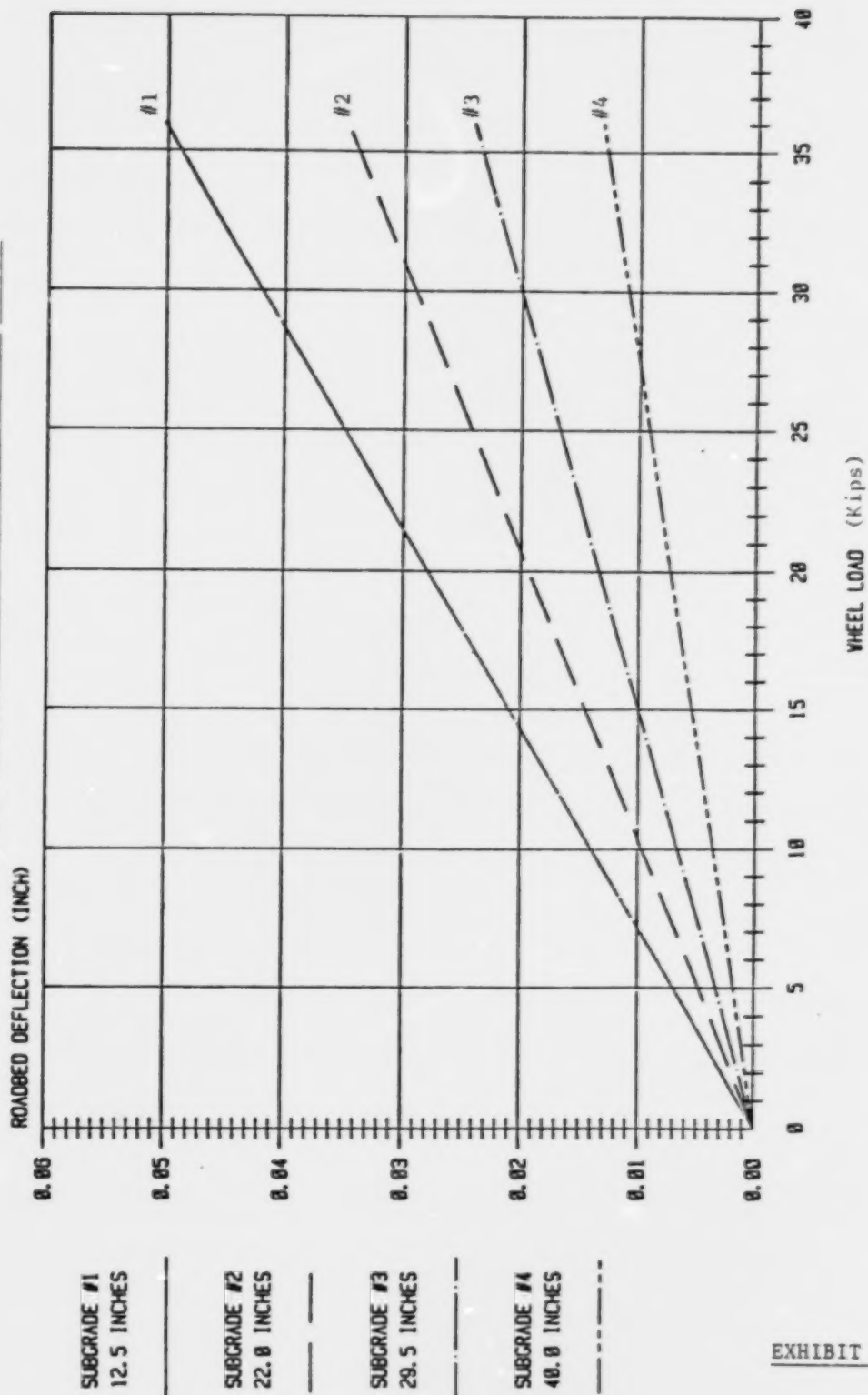
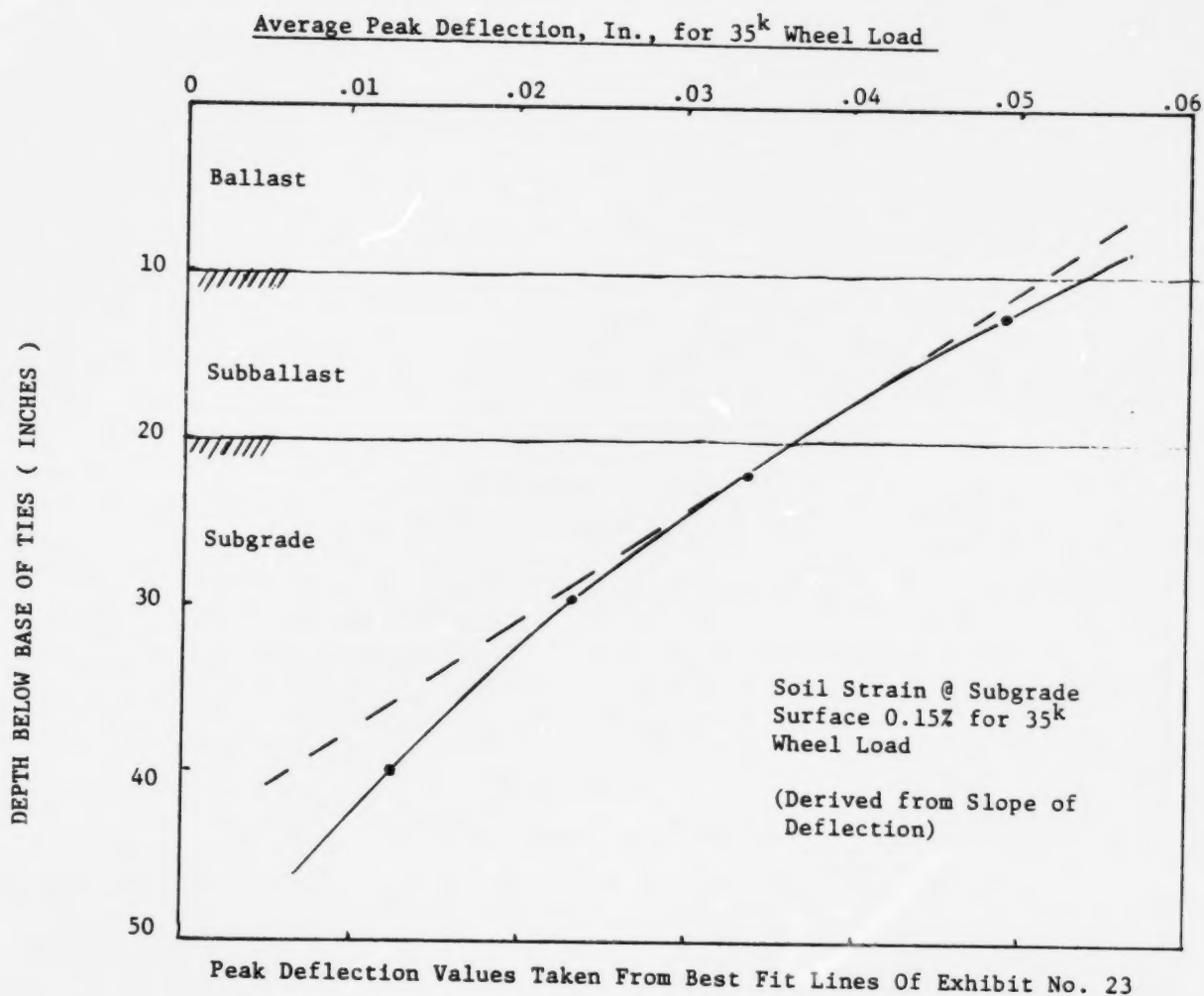


EXHIBIT NO. 23

AVERAGE ROADBED DEFLECTIONS FOR 35 KIP WHEEL LOADSITE NO. 18 MILE 118.2 JOLIETTE SUB. AUG. 21/80

3. RAIL AND SUBGRADE SETTLEMENT RATES

3.1 Measurement Techniques

All of the roadbed elastic deflection and subgrade strain data presented in this and previous Reports (see Bibliography) showed correlation with depth of granular, but there were no clear-cut criteria which distinguished between acceptable and unacceptable limits. Further information was required on the rate of track surface deterioration or, in other words, the track maintenance requirement.

Over a three-year period (1980-1982), rail and subgrade settlements have been measured at two adjacent Montreal area sites, and subgrade settlements only have been measured at five different Edmonton area sites. All of the track settlements have been plotted with time, and graphs are presented in Appendices A, B, and C.

Appendix A - Rail and Subgrade Settlement
Site No. 18, Mile 118.2 Joliette Sub.

Appendix B - Rail and Subgrade Settlement
Site No. 19, Mile 122.3 Joliette Sub.

Appendix C - Subgrade Settlement only,
Sites 11, 14, 15, 16, 17 Edmonton Area

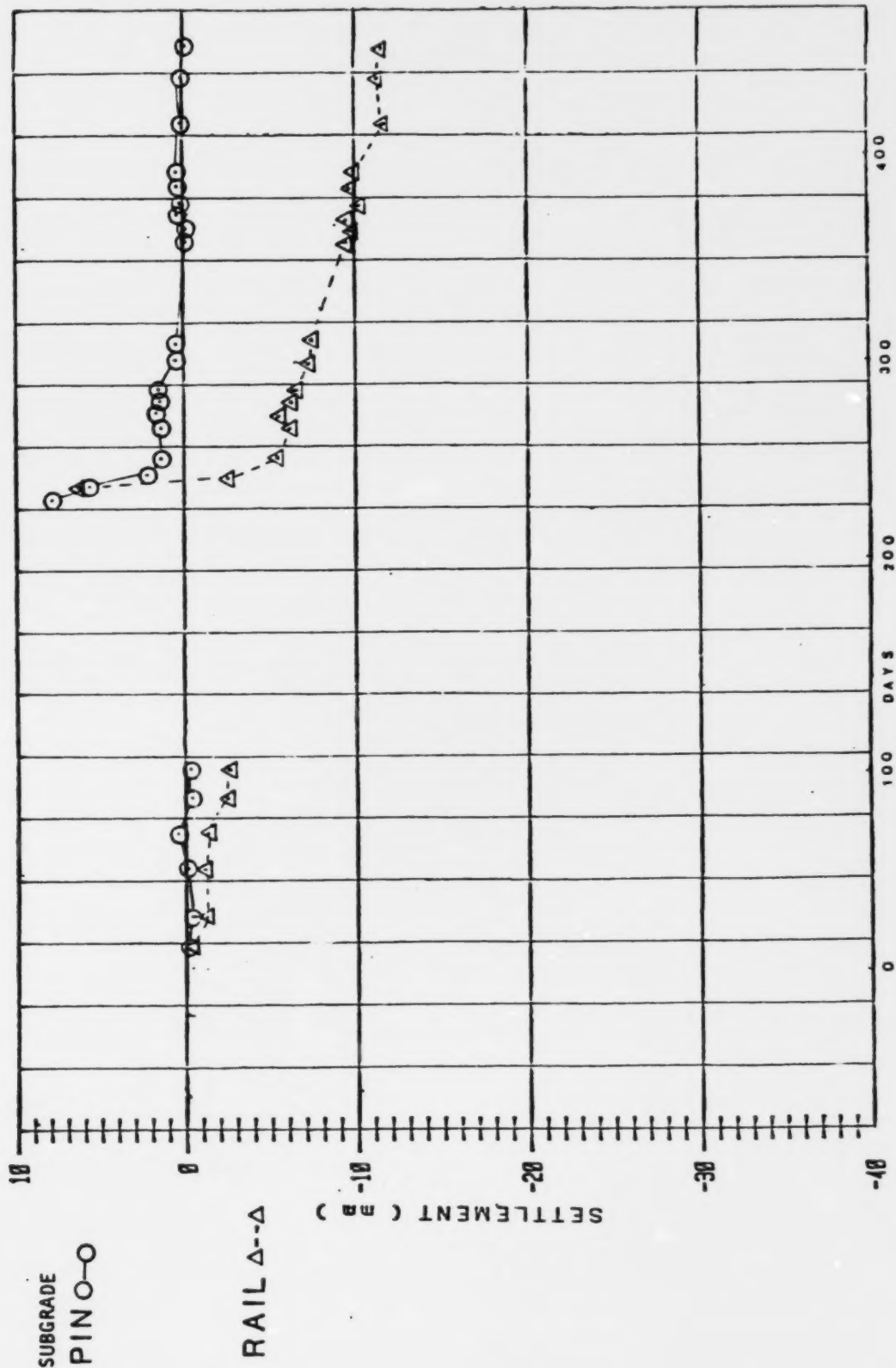
On the two Joliette Sub. sites, rail and subgrade settlements were measured with precise level (reading accuracy 0.01 mm), referenced to benchmark rods located roughly 20 ft. from track. On the Edmonton area sites, subgrade settlements were measured with a dial gauge (reading accuracy 0.001 in.), referenced to ten-foot benchmark rods which were driven concentric through pipes welded to the settlement plates. The benchmark rods were in plastic casings for the top 7 ft. in order to isolate them from the roadbed surface settlements. It is not known if a 10 ft. benchmark located right beside the rail is long enough to provide an absolute reference, but certainly the subgrade settlements at 10 ft. depths would be small.

3.2 Discussion of Sample Settlement Graphs

Sample settlement graphs for Site No. 18 are shown in Exhibits No. 25 (a) and (b). Settlement rates have been established by drawing straight lines through the plotted points. It will be noted that there was considerable frost heave at this site, and that there were no excessive subgrade settlements during the spring thaw. In early June of 1982, ballast was added to the track and ties were lifted and manually tamped (in order not to damage buried instrumentation). Settlement rates before and after this surface lift have been established. It will be noted that there were increased and non-linear rail settlements immediately following the lift and then settlement became linear again.

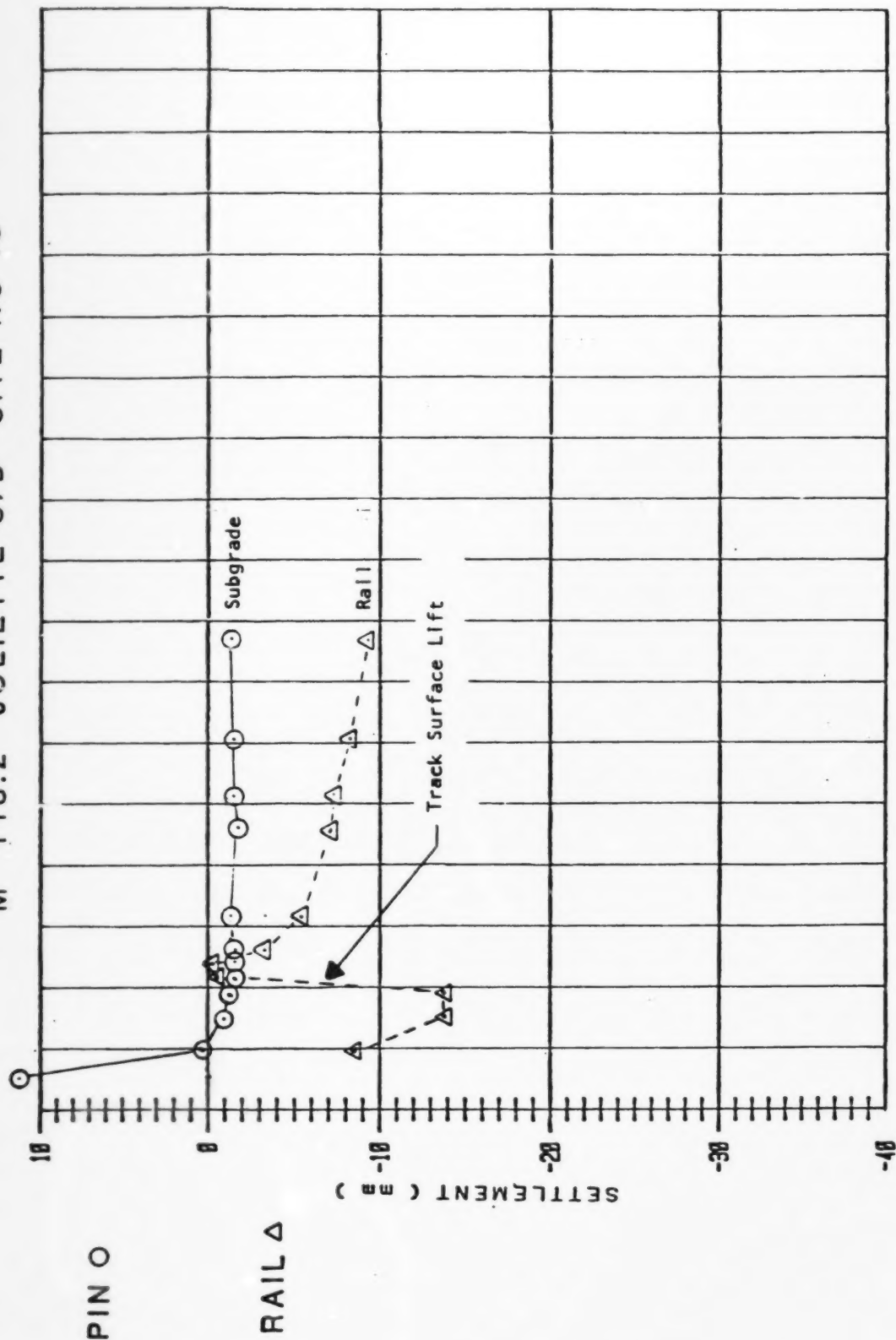
SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-A



SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18



APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

MONTHS

1962

Sample settlement graphs for Site No. 19 are shown in Exhibits No. 26 (a) and (b). It will be noted that there was considerable frost heave, but that there was no excessive subgrade settlement during spring thaw. The settlement rate was linear and continued in spring from where it had stopped the previous fall. In June of 1981, ballast was added to the track and ties were lifted roughly 7 cm and machine tamped. Rail settlement rates prior to and following this mechanized track surface lift have been established. It will be noted that there was rapid and non-linear settlement immediately following the surface lift but that within two months a uniform settlement rate had been established. Settlement rates in this Report refer to the uniform or linear settlements. The non-linear or stabilizing settlements are dealt with in a later section of the Report.

Subgrade settlements for Site No. 14, Mile 9.67 Edson Sub. (just west of Edmonton) have been shown in Exhibits No. 27 (a) and (b). This location had 40 in. depth of granular below tie base and subgrade settlement rates were low.

All of the settlement graphs have been included in the Appendices, and, although the Report becomes bulky, it will be restated here that a few well documented track measurements are worth a dozen unvalidated theories.

While settlement rates with time are useful to correlate directly with track maintenance requirement, it is traffic density which governs the rate of settlement at any particular test site.

3.3 Traffic Density at Test Sites

The annual traffic density in million gross tons (MGT) per year has been listed in Table No. 8 and plotted in Exhibit No. 28 for all the test sites over the past 10 years. At time of Report writing the traffic density reports for 1982 had not yet been prepared, and 1982 traffic has been assumed identical to 1981 traffic for correlating 1982 settlement rates. In 1979, double tracking was progressing west of Edmonton and this accounts for the sudden halving of traffic density at Site No. 14. It will be noted that traffic density ranged from 11 to 50 MGT per year at the various test sites.

3.4 Settlement Rates Based on Time and Traffic

The subgrade and rail settlement rates for the twenty measurement stations at Site No. 18 Joliette Sub. have been tabulated in Table No. 9. The ballast depth has been listed as 12 in. but it was difficult to establish the dividing line between ballast and sub-ballast on this old track. The subgrade settlement plates were installed at depths below base of tie ranging from 19 to 31-1/2 in. Again, it was difficult to establish a clear dividing line between the sandy subballast and top of sandy silt subgrade, but the

SETTLEMENT GAUGES

JOLIETTE SUBDIVISION

SITE NO. 19 GAUGE NO. 2

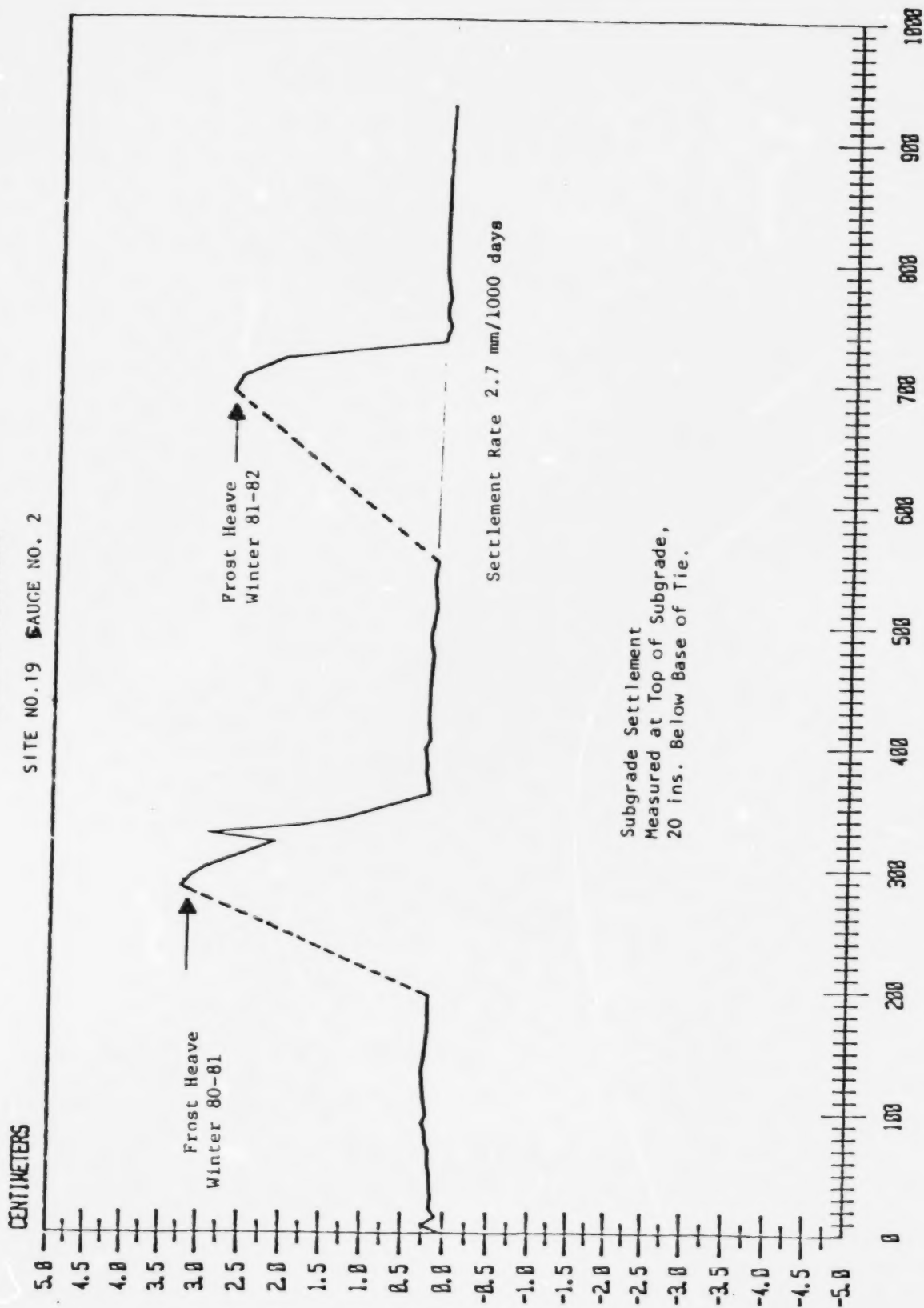


EXHIBIT NO. 26 (a)

RAIL SETTLEMENT

STATION 4 MILE 122.3 JOLIETTE S/D

SITE NO. 19

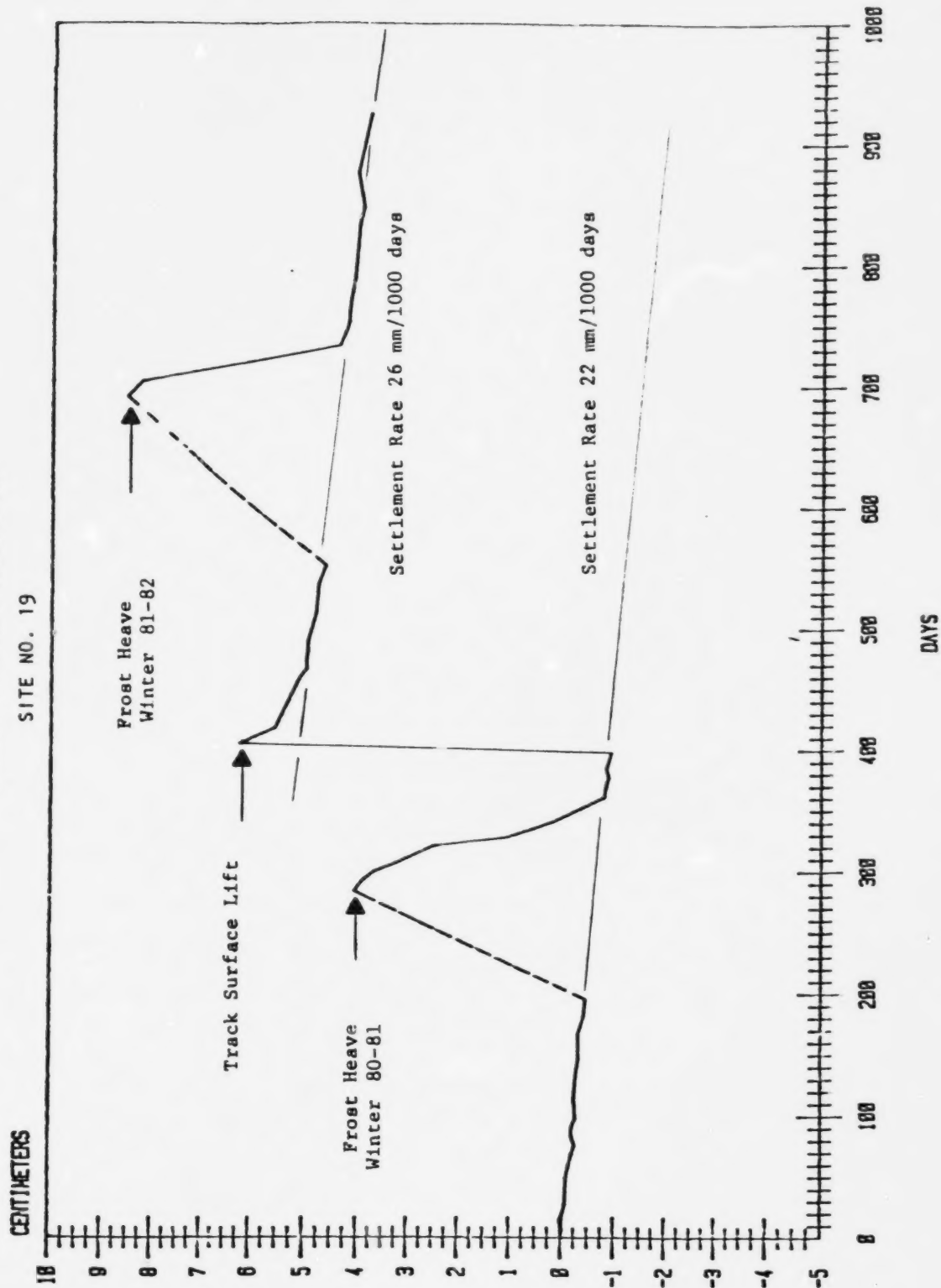
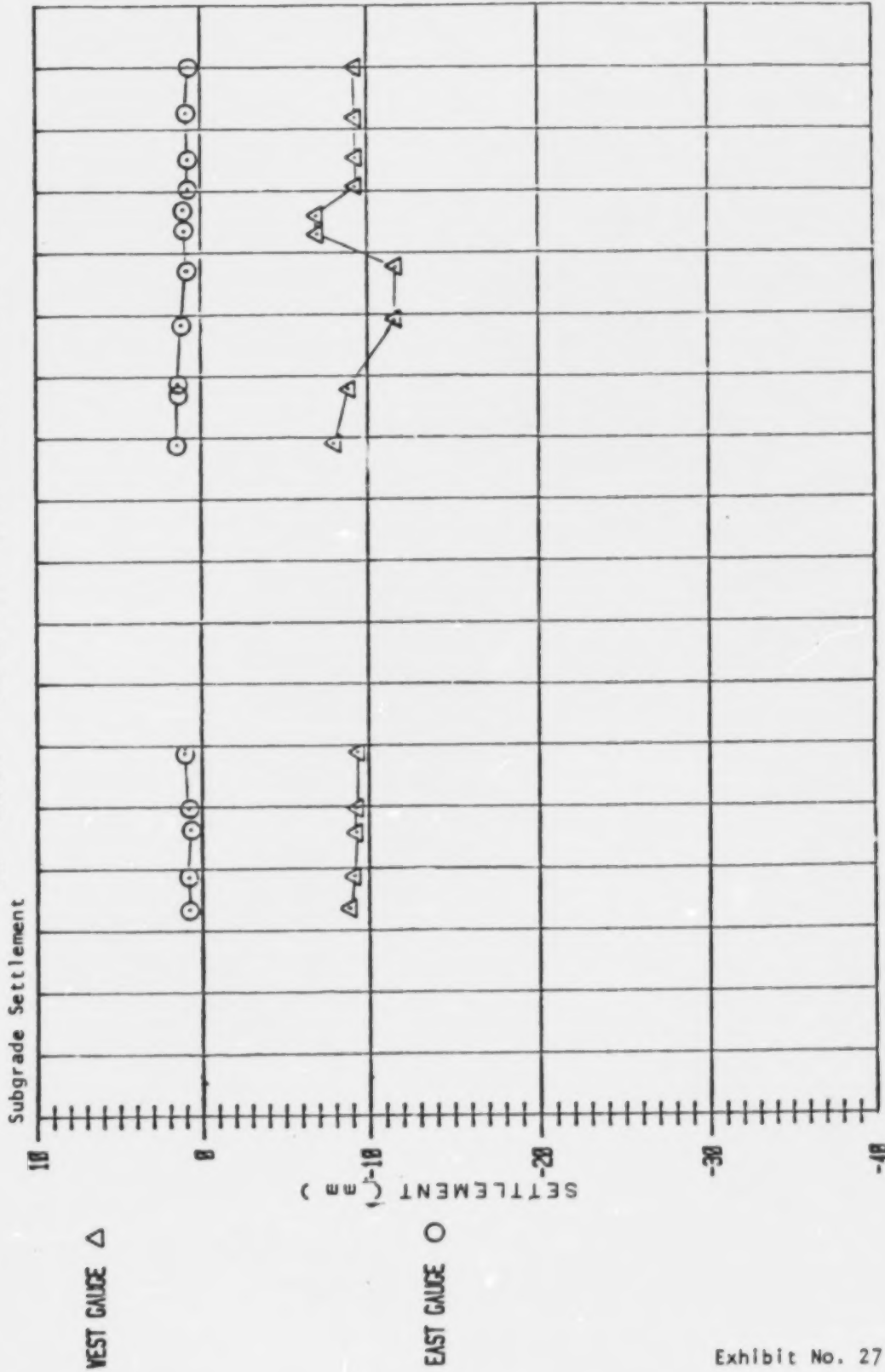


Exhibit No. 26 (b)

SETTLEMENT GAUGE READINGS

M 9.67 EDSON S/D SITE NO 14



JUN JUL AUG SEP OCT NOV DEC

MONTHS 1988 - 1989

SETTLEMENT GAUGE READINGS

M 9.67 EDSON S/D SITE NO 14

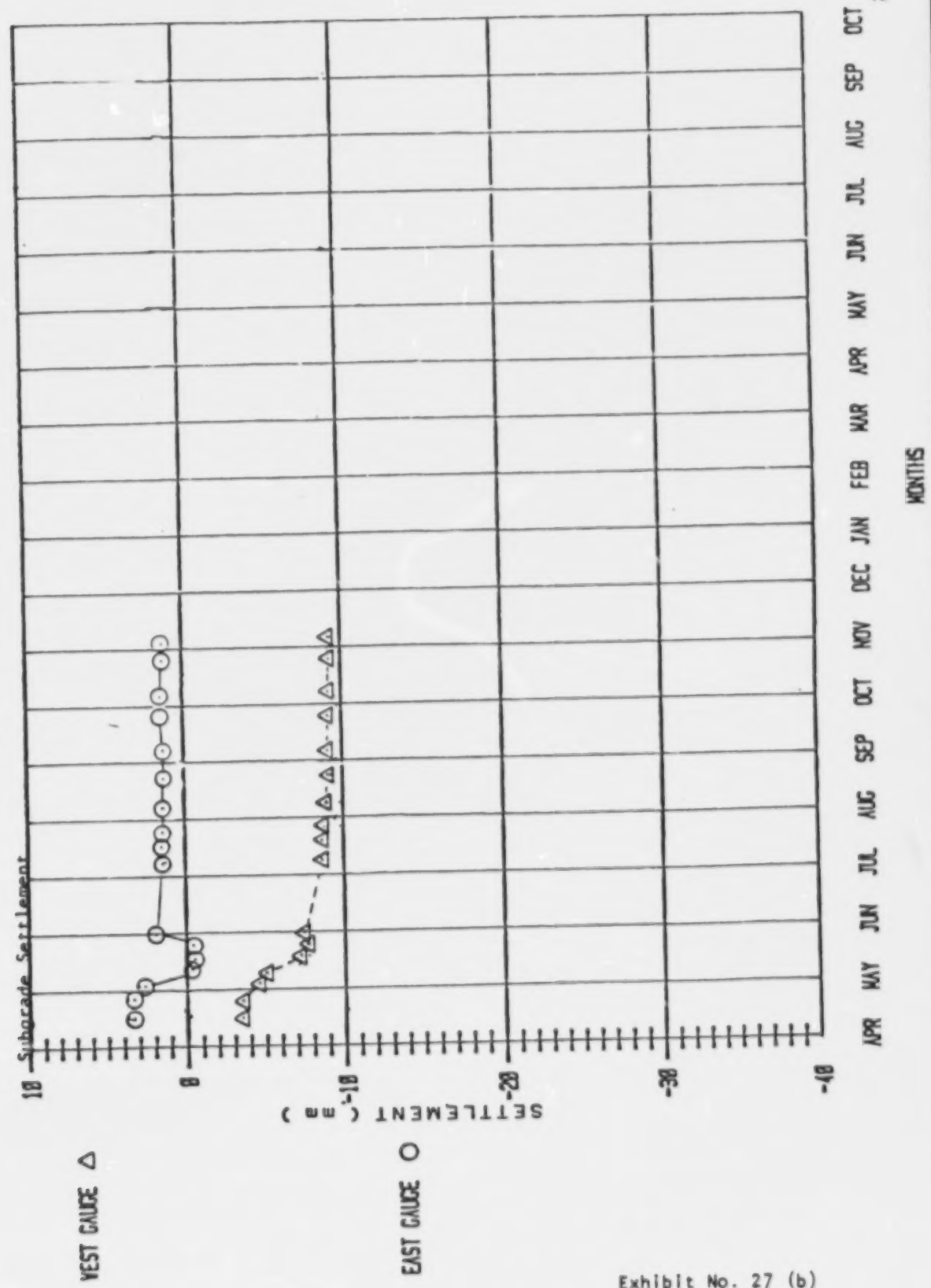


TABLE NO. 8

ANNUAL TRAFFIC DENSITY
AT THE BALLAST THICKNESS TEST SITES
TRAFFIC IN MILLION GROSS TONS/YEAR MGT

Year	Site #11	Site #14	Site #15	Site #16	Sites #18 & #19
1970	18.785	23.992	23.992	5.091	9.528
1971	22.462	29.134	29.134	6.199	9.365
1972	24.748	31.529	31.529	6.532	9.877
1973	24.015	30.558	30.558	6.408	9.650
1974	25.109	33.153	33.153	8.813	10.890
1975	25.231	30.564	30.564	8.769	10.861
1976	25.241	31.899	31.899	7.805	10.542
1977	27.396	36.275	36.275	8.866	10.586
1978	30.644	40.894	40.894	9.144	11.327
1979	32.621	* 22.360	* 44.719	12.013	11.612
1980	36.797	* 25.303	* 50.607	12.475	10.938
1981	34.518	* 25.233	* 50.465	12.805	11.369
<u>1980</u> <u>1970</u>	1.96	1.05	2.11	2.45	1.15

* Corrected for double traffic.

Site #11 - Mile 257.8 Wainwright Sub.
 Site #14 - Mile 9.67 Edson Sub. (double-tracked in 1979)
 Site #15 - Mile 38.7 Edson Sub.
 Site #16 - Mile 123.4 Vegreville Sub.
 Site #18 & #19 - Mile 118.2 and 122.3 Joliette Sub.

ANNUAL TRAFFIC DENSITY

At The Ballast Thickness Test Sites

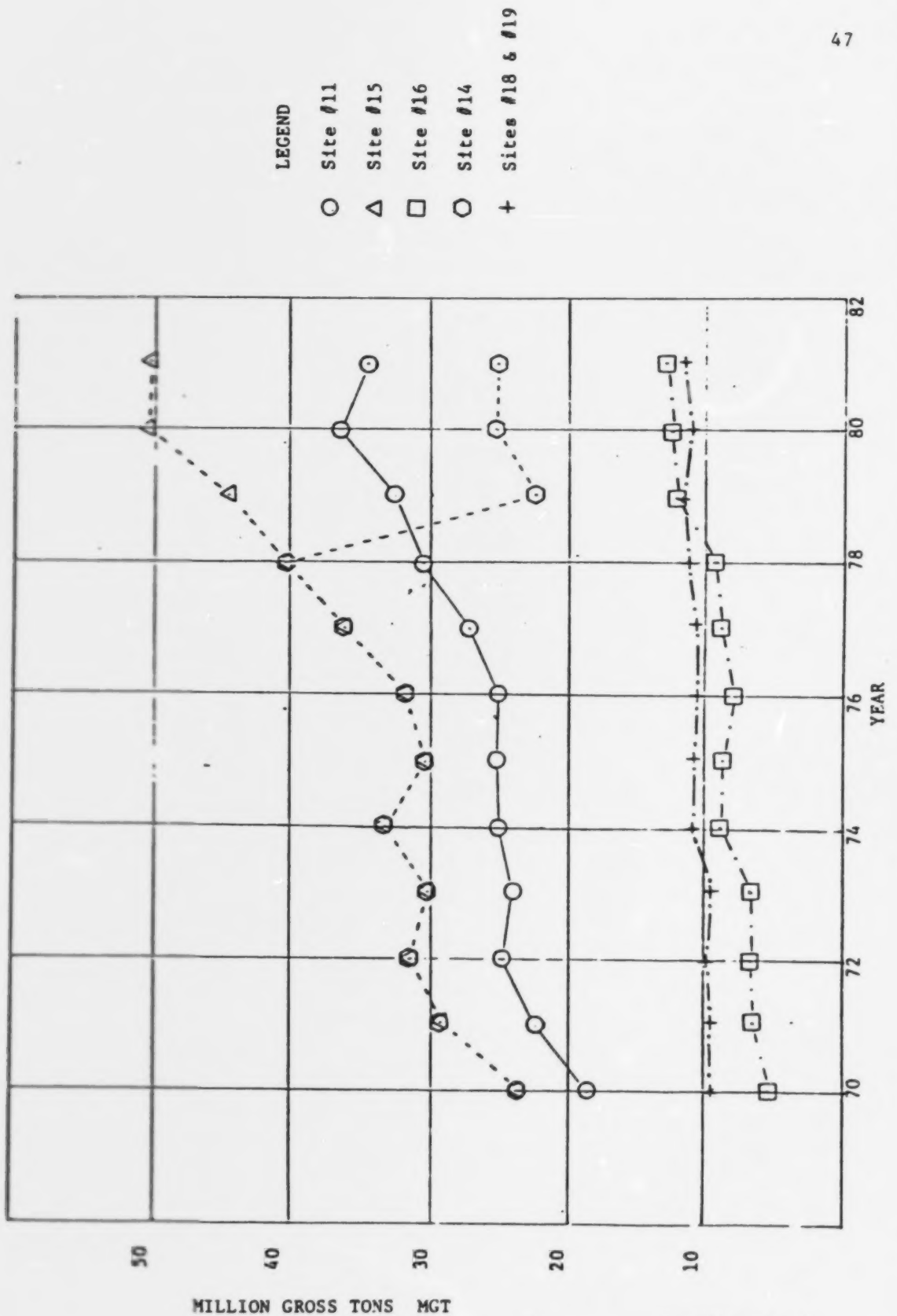


TABLE NO. 9

SUBGRADE AND RAIL SETTLEMENT RATES

SITE NO. 18 MILE 118.2 JOLIETTE SUB.

STATION (SEE EXHIBIT NO. 2)	DEPTH BELOW BASE OF TIE INCHES		SETTLEMENT RATE							
			1980 - 1981				1982			
			mm/MONTH		mm/MGT		mm/MONTH		mm/MGT	
	BALLAST*	SUBGRADE	SUBGRADE	RAIL	SUBGRADE	RAIL	SUBGRADE	RAIL	SUBGRADE	RAIL
A	12	26	0.22	1.05	0.24	1.16	0.03	0.71	0.033	0.782
B	12	31.5	0.22	0.22	0.24	0.24	0.02	0.61	0.022	0.672
C	12	23.5	0.22	0.89	0.24	0.96	0.06	0.50	0.066	0.551
D	12	23.5	0.19	0.89	0.21	0.96	0.01	0.50	0.011	0.551
E	12	19	0.31	0.83	0.34	0.92	0.10	0.44	0.110	0.485
F	12	23	0.17	0.81	0.18	0.89	0.12	0.46	0.132	0.507
G	12	21	0.33	0.97	0.37	1.07	0.10	0.54	0.110	0.594
H	12	22.5	0.33	0.81	0.37	0.89	0.14	0.52	0.154	0.573
I	12	24	0.19	0.50	0.21	0.55	0.16	0.40	0.176	0.441
J	12	28.5	0.39	1.02	0.43	1.13	0.06	0.56	0.066	0.617
K	12	23.5	0.14	1.39	0.15	1.53	0.28	0.37	0.308	0.407
L	12	22.5	0.19	1.44	0.21	1.59	0.21	1.17	0.231	1.289
M	12	25	0.33	0.94	0.37	1.04	0.19	1.31	0.209	1.443
N	12	25.5	0.19	0.86	0.21	0.95	0.18	0.93	0.198	1.024
O	12	25	0.28	1.00	0.31	1.10	0.25	0.92	0.275	1.013
1B	12	21	0.19	0.83	0.21	0.92	0.18	1.14	0.198	1.256
1E	12	19	0.22	0.78	0.25	0.86	0.17	1.02	0.187	1.123
1H	12	24	0.08	0.78	0.09	0.86	0.26	1.07	0.286	1.178
1K	12	22	0.16	1.17	0.18	1.28	-	0.69	-	0.760
1N	12	24.5	0.42	0.83	0.46	0.92	0.39	0.58	0.429	0.639
AVERAGE	12	23.7	0.238	0.901	0.264	0.993	0.153	0.722	0.168	0.795

* RANGED FROM 8 - 14 INCHES, DIFFICULT TO DISTINGUISH DIVIDING LINE BETWEEN BALLAST AND SUBBALLAST, NOMINAL BALLAST DEPTH TAKEN AS 12 INCHES.

settlement plates were essentially located on the nominal top of subgrade. Settlement rates have been listed in mm/month and mm/MGT for 1980-81 (before the track surface lift) and for 1982 (following the track surface lift). The lift averaged about 5/8 in. with manual tamping.

The settlement rates were quite erratic over this 100 ft. length of track and this indicates maintenance requirement. It is non-uniform settlement which causes track surface roughness and loss of geometry. During 1980-81, 26% of the total average settlement occurred in the subgrade and 74% occurred in the granular (ballast and subballast) layers. The average rail settlement was 0.901 mm/month or 10.8 mm (0.42 in.) per year, and this is considered relatively high for a track with traffic density of only 11 MGT/year. If traffic were to suddenly increase to 50 MGT/year on this line, rail settlements would soar to 2 in. per year and it would be very difficult if not impossible to maintain track surface. It is the objective of this study to determine how much more granular depth would be required to keep track maintenance at acceptable levels.

During 1982, following the small manual surface lift at Site No. 18, the subgrade and rail settlement rates were lower by 35 and 20% respectively than they had been in 1980-81. It is not felt that the small lift itself actually had such beneficial results. It may be that traffic density had dropped on account of the recession but the 1982 traffic figures are not yet available (1982 traffic was assumed identical to 1981 traffic). In 1982, 21% of the total average settlement occurred in the subgrade and 79% occurred in the granular layer.

All the subgrade and rail settlement rates given above were determined by drawing straight lines through the settlement points. Following the surface lift, there were increased and non-linear settlements of the rail before uniform settlement rates were re-established. The accelerated settlements will be dealt with in a following section of this Report.

The subgrade and rail settlement rates at Site No. 19 on the Joliet Sub. are listed in Table No. 10. During 1980 and on to spring of 1981, subgrade settlement rates were nil while rail settlement rates averaged 0.59 mm/month. Granular depth was 23 in. below base of tie. Following the mechanized track surface lift in June of 1981 and with granular depth now almost 26 in., subgrade settlement rates were 0.102 mm/month while rail settlement rates had increased to 0.72 mm/month (an increase in settlement rate of 22%). The zero subgrade settlement rate in 1980 cannot be explained but that is what was recorded on the three settlement plates. The 2-1/2 to 3 in. of additional ballast actually increased the subgrade settlement rate. Research work is sometimes frustrating when measurements do not make sense and no plausible explanations can be found. The rail settlements did make sense and it is clear that the mechanized surface lift caused increased rates of track settlement (over and beyond the rapid stabilizing settlements that occurred within the first two months).

TABLE NO. 10

SUBGRADE AND RAIL SETTLEMENT RATES

SITE NO. 19 MILE 122.3 JOLIETTE SUB.

SUBGRADE GAUGE	RAIL STATION	DEPTH BELOW BASE OF TIE INCHES		SETTLEMENT RATE							
				1980 - 1981				1981 - 1982			
				mm/MONTH		mm/MGT		mm/MONTH		mm/MGT	
		BALLAST	SUBGRADE	SUBGRADE	RAIL	SUBGRADE	RAIL	SUBGRADE	RAIL	SUBGRADE	RAIL
	1				0.63		0.69		0.63		0.69
	2				0.60		0.66		0.78		0.86
1	3	-12*	23	-0	0.63	-0	0.69	0.123	0.75	0.135	0.83
2	4		23	-0	0.66	-0	0.73	0.081	0.78	0.089	0.86
3	5		23	-0	0.69	-0	0.76	0.102	0.69	0.112	0.76
	6				0.63		0.69		0.72		0.79
	7				0.45		0.49		0.78		0.86
	8				0.45		0.49		0.60		0.66
AVERAGE					0.53		0.65	0.102	0.72	0.112	0.79

NOTE: TRAFFIC 0.908 MGT/MONTH.

* RANGED FROM 9 - 14 INCHES, DIFFICULT TO DISTINGUISH DIVIDING LINE BETWEEN BALLAST AND SUBBALLAST.

This raises some questions. If additional granular depth is supposed to decrease track maintenance requirement, how can increased track settlement rates be explained? Would ballast compaction following the mechanized lift have stabilized the new ballast? While these questions cannot be answered from the available data, it is clear that precise track elevation measurements taken at frequent intervals over a 2- or 3-year period could provide much needed information.

Subgrade settlement rates for the five Edmonton area test sites are listed in Table No. 11. Two or three settlement plates were installed at each site during 1980. Some of the rods were struck and destroyed by track maintenance machinery and that accounts for some of the missing data. All of the plates installed on the Vegreville Sub. in 1980 were destroyed and three new plates were installed in 1981. In general, settlement rates at the various test sites were erratic, indicating loss of track geometry (no rail settlement measurements were taken at these sites). On the west settlement plate at Site No. 11, Mile 257.8 on the Wainwright Sub., the 1981 settlement rate was roughly 10 times greater than on the other plates, and this value has been excluded from the average (reason for excessive settlement not known).

In order to bring the subgrade settlement rates into perspective, estimated track maintenance requirements have been presented in Table No. 12. It has been assumed that the track maintenance effort required with time is directly proportional to the track subgrade settlement rate per month. This assumption may be completely invalid and has not been correlated with actual track maintenance requirements, but it serves to illustrate some important factors. The track with 40 in. granular depth on the Edson Sub. has been taken as reference for illustration purposes (this track had the lowest subgrade settlement rate). If we give this site a track maintenance factor of 1.00, then 57% more maintenance effort would be required on the Wainwright Sub. to maintain similar track conditions, and 5.58 times more effort would be required on the Vegreville Sub. Because of different traffic conditions (density and speed), tracks are not maintained to the same levels and the above factors do not directly apply. On the Calder Yard bypass, which was new track construction opened to traffic in 1980, 2.31 times more maintenance effort would be required than on the Edson reference site. The estimated track maintenance factors have been included for illustration purposes only - they may not be correct but they highlight significant differences between test sites. Before the final close-out Report on this study is written, an effort will be made to determine actual track maintenance requirements at each of the test sites.

No assumptions have to be made when looking at the subgrade settlement rates in mm/MGT. If traffic on all test sites were increased to 50 MGT/year, then the two Edson sites and the Wainwright site would have 4-5 mm subgrade settlement/year while the Vegreville site would have roughly 50 mm/year (a factor of 10). It would not be practical to try and maintain such a track, and additional granular depth would be required.

TABLE NO. 11

SUBGRADE SETTLEMENT RATES - WESTERN CANADA

SITE NO.	SUBDIVISION	MILE	GAUGE DEPTH, IN.	YEAR	TRAFFIC MGT	SUBGRADE SETTLEMENT RATE	
						mm/MONTH	mm/MGT
11	WAINWRIGHT	257.8	EAST 27	1980 1981 1982	36.797 34.518	0.123 0.364	0.040 0.127
			MIDDLE 27	1980 1981 1982	34.518 34.518	0.556 0.219	0.193 0.076
			WEST 27	1980 1981 1982	36.797 34.518 34.518	0.216 (2.540)* 0.289	0.070 (0.883)* 0.100
			AVERAGE			0.295	0.101
14	EDSON	9.67	EAST 40	1980 1981 1982	25.303 25.233 25.233	0.178 0.200 0.182	0.084 0.095 0.087
			WEST 40	1980 1981 1982	25.233 25.233	0.200 0.182	0.095 0.087
			AVERAGE			0.188	0.090
15	EDSON	38.7	EAST 29	1980 1981 1982	50.607 50.465 50.465	0.174 0.410 0.462	0.041 0.098 0.110
			WEST 29	1980 1981 1982	50.607 50.465 50.465	0.290 0.435 0.214	0.069 0.103 0.051
			AVERAGE			0.331	0.079
16	VEGREVILLE	123.4	A 17	1980 1981 1982	12.805	2.110	1.978
			B 17	1980 1981 1982	12.805 12.805	1.050 0.213	0.984 0.200
			C 17	1980 1981 1982	12.805	0.833	0.781
			AVERAGE			1.05	0.986
17	CALDER YARD BYPASS		EAST 27	1980 1981	7 7	0.373 0.204	
			MIDDLE 27	1980 1981	7 7	0.285 0.545	
			WEST 27	1980 1981	7 7	0.766 0.433	
			AVERAGE			0.434	

* EXCLUDED FROM AVERAGE, REASON FOR HIGH SETTLEMENT RATE NOT KNOWN.

TABLE NO. 12

ESTIMATION OF TRACK MAINTENANCE REQUIREMENTS

Site No.	Subdivision	Depth of Granular, In.	Average Subgrade Settlement mm/Month	Estimated Track Maintenance Factor	Subgrade Settlement	
					mm/MGT	mm/50 MGT
11	Wainwright	27	0.295	1.57	.101	5.05
14	Edson	40	0.188	1.00*	.090	4.50
15	Edson	29	0.331	1.76	.079	3.95
16	Vegreville	17	1.05	5.58	.986	49.3
17	Calder Yard Bypass	27	0.434	2.31	-	-

* Taken as reference for illustration purposes.

The unevenness of the subgrade settlement rates at Site No. 18 Joliette Sub. are shown in Exhibit No. 29. Over the 100 ft. test length, settlement rates varied from almost nil to 0.4 mm/month. The settlement rates in 1982 were clearly lower than they had been in 1980-81, except at the west end of the test site where several of the ties were pumping mud (stations L to N). Several ties within 300 ft. of the east end of the site were also pumping mud, indicating that the granular depth was insufficient in localized spots (where settlement plates were installed, the minimum granular depth was 19 in.).

In Exhibit 29 (b), subgrade settlement rates have been plotted against depth of subgrade below base of tie. It has been stated that it was difficult to distinguish the dividing line between the sandy subballast and the subgrade, and the twenty settlement plates were installed at depths ranging from 19 to 31-1/2 in. There was no clear relationship between depth of granular and subgrade settlement rate at this site. One would have expected lower settlements from the deeper plates but such was not the case - another frustrating set of results.

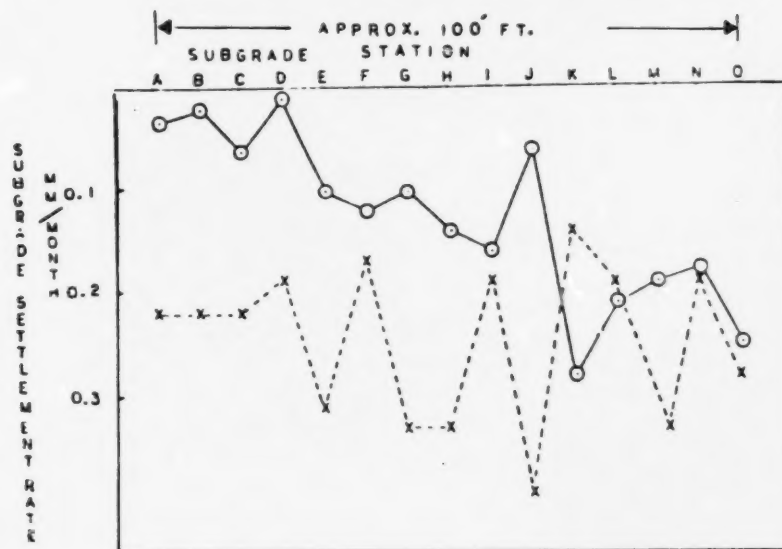
The average subgrade settlement rates for all the test sites have been plotted against the average depth of granular in Exhibit No. 30. In the (a) graph, a smooth curve has been drawn through the mm/month settlement points for the five Edmonton sites and the Joliette sites have not been fitted to the curve because of the different soil types. The greater depths of granular show significantly lower settlement rates, and, in general, less spread in the data. Both of these factors mean lower track maintenance requirement. It would be difficult to select an "optimum" depth of granular from the (a) graph because of the smooth curve, and it might also prove misleading because of the different traffic densities and track maintenance standards at the various sites.

In Exhibit No. 30 (b), subgrade settlements in mm/MGT have been plotted against depth of granular. While four Edmonton area data points do not provide conclusive evidence, the graph indicates that 30 in. of granular provided "maximum effective" construction depth for Lake Edmonton clay. Clearly 17 in. of granular was far too little, and the extra 10 in. of granular in going from 30 to 40 in. depth provided no return on investment. The settlement rate for Joliette Site 18 fell on the Edmonton clay curve while settlements were lower at Joliette Site 19. The Joliette soils were stronger than the Edmonton soils and a subsequent Report will document all the laboratory test data generated by McGill Geotechnical Research Centre.

It is considered that Exhibit No. 30 (b) represents the heart of this Report. It indicates granular depth requirement as a function of traffic. This chart was generated by taking a series of simple dial gauge readings and elevations over a 3-year period. The graph stands alone, independent of types of ballast, subballast,

SUBGRADE SETTLEMENT RATES

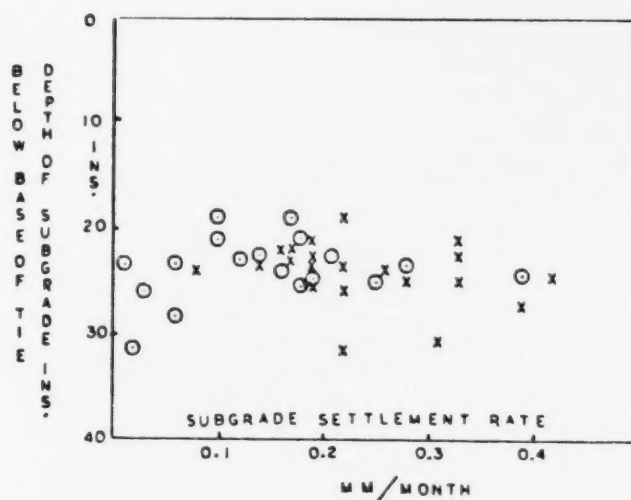
Site No. 18 Mile 118.2 Joliet S/D



x 1980-81

o 1982

- a) Subgrade Settlement Variation with Distance along Track.

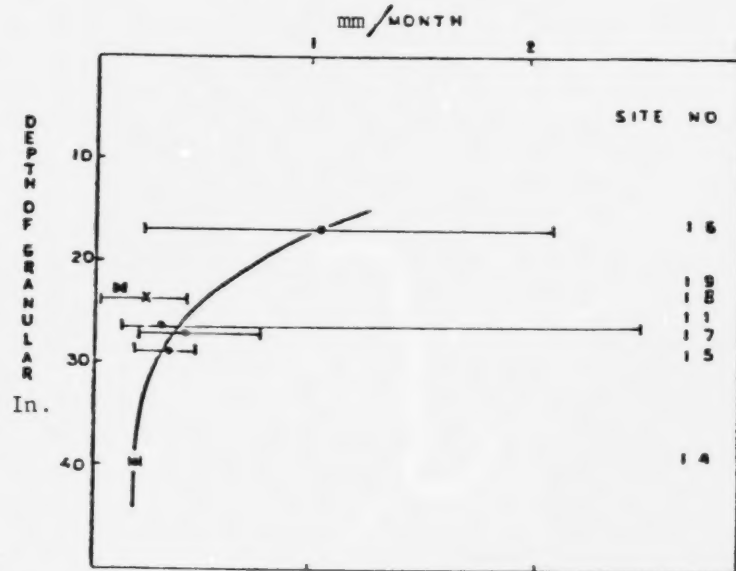


x 1980-81

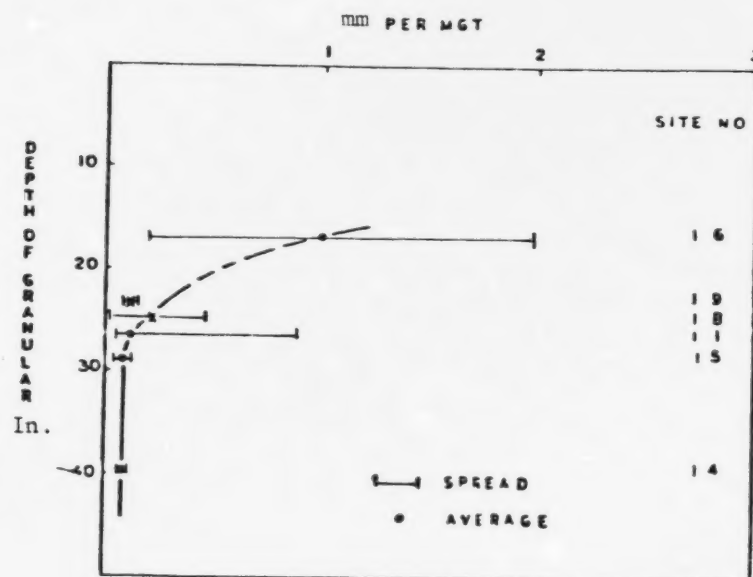
o 1982

- b) Subgrade Settlement Variation with Depth of Granular.

SUBGRADE SETTLEMENT RATE vs. DEPTH OF GRANULAR



a) Subgrade Settlement Rate mm/Month



b) Subgrade Settlement Rate mm/MGT

and subgrade, tie condition, rail weight, and wheel loading spectra, or in other words, all of the parameters necessary for theoretical roadbed design. The only outside information required was the traffic density report.

The authors feel that a proven case cannot be made from only four data points, but that a simple means for recording important data has been established. CN Rail Research will not be expanding upon this study at the present time because of other high priority projects, but it is strongly recommended that any other interested parties pursue this method of measurement. If two people were assigned full time for two years, they could probably install settlement plates and take readings at 50 test sites. Depths of subgrade should range from 15 to 40 in. and different soil types should be investigated. Rail elevations should also be recorded because it is track surface loss of geometry that dictates maintenance requirement. Many other factors such as rail weight, tie type, size and spacing, ballast type and gradation, subballast type and gradation, and subgrade soil properties could be recorded to help explain possible variations in data, but it should be stressed that the subgrade and rail settlements are the overriding important parameters. Two years of field measurement could provide better information than the last fifty years of roadbed design theories.

While the above approach would constitute a research fact finding mission, it would take two years to establish good data. If one were faced with upgrading hundreds of miles of track for projected increased traffic densities, it would not be practical to spend two years taking dozens of settlement readings nor would it be practical to make hundreds of roadbed digs to establish the existing but variable granular depths. What would be practical would be the measurement of rail elastic deflections or in essence the track modulus, provided that the elastic deflections could be correlated with other roadbed settlement data. To determine if roadbed elastic deflection correlates with roadbed settlement rate, it would require that both sets of measurements be taken during the two-year research phase. If such correlation was possible, elastic deflections could subsequently be used to determine granular depth requirement.

3.5 Effectiveness of Track Surface Lifts

The effectiveness of a small track surface lift with manual tamping has been documented in Table No. 13. The short section of track at Site No. 18 Joliette Sub. was hand-tamped to avoid damaging buried instrumentation. The rail lift varied from 2.5 to 26 mm and averaged 14.4 mm (0.57 in). As the new and disturbed ballast stabilized, non-linear rail settlements occurred over the first few weeks until a uniform rate of settlement was established. On average, 63% of the lift was lost within the first 2.1 months, and at the now stabilized average settlement rate of 1.08 mm/month, the entire lift would have been dissipated within 0.94 years. This corresponds to only 10.7 MGT of traffic, and demonstrates the temporary nature of manual tamping maintenance.

TABLE NO. 13

EFFECTIVENESS OF TRACK SURFACE LIFT (HAND TAMP)

SITE NO. 18 MILE 118.2 JOLIETTE SUB.

Station No.	Rail Lift mm	Loss of Lift			Rail Settlement mm/18 Mo.	Lift Life Years
		mm	%	Months		
A	14	7	50	2.5	12.8	1.03
B	10.5	7.3	70	2.5	11.0	0.64
C	5.5	5.5	100	1.4	9.0	0.12
D	5	5	100	1.0	9.0	0.08
E	7	7	100	2.0	8.0	0.16
F	7	7	100	1.8	8.3	0.15
G	4.4	4.4	100	1.2	9.8	0.10
H	2.5	2.5	100	1.2	9.4	0.10
I	3.8	3.8	100	1.8	7.2	0.15
J	10.8	10.8	100	9.0	10.0	0.75
K	20.1	12.0	59.7	3.0	6.7	2.06
L	25.8	7.4	28.7	1.2	21.0	1.41
M	24.6	6.3	25.6	1.3	23.5	1.28
N	20.6	5.6	27.2	0.8	16.7	1.41
O	17.6	5.7	32.4	1.2	16.5	1.18
1B	25.4	6.6	26.0	1.3	20.5	1.48
1E	14.0	6.0	42.9	1.3	18.3	0.76
1H	13.3	4.7	35.3	1.3	19.3	0.78
1K	29.3	11.5	39.2	2.7	12.5	2.36
1N	26.1	7.8	29.9	2.8	10.5	2.84
Average	14.4	6.7	63.3	2.1	13.0	0.94

At Site No. 19 Joliette Sub., the track was lifted an average of 70 mm (2.75 in.) and mechanically tamped. As shown in Table No. 14, on average 11.4 mm or 16% of the lift was lost within 2-1/2 months, and at the new stabilized settlement rate of 0.73 mm/month, the entire lift would have been dissipated within 7 years. This corresponds to 79 MGT of traffic at this location.

The projected loss of track surface lift with time has been documented for the two test sites in Exhibit No. 31. A lift of 3/8 in. was lost almost immediately, a lift of 1 in. would dissipate in 2 to 3 years, and a lift of 3 in. would dissipate in 6 to 8 years. Measurement of rail settlements should provide a simple means of evaluating the effectiveness of track surfacing and ballast compacting equipment and therein lies a possible future research project.

Exhibit No. 32 shows some concrete tie settlement measurements taken on the CN Rail Research test track in 1976 and 1977. This track had nonconsolidated subgrade, virtually noncompacted ballast, and 24 in. of granular below base of tie. Traffic consisted of a loaded 100-ton car cycled back and forth at 2 mph. Tie (rail) settlement rates ranged from 2 to 4 mm/MGT. These settlement rates ranged up to 40 times greater than those recorded on the Edson Sub., and clearly the Research Centre test track could not even begin to cope with mainline type traffic. Granular depth and roadbed consolidation were two key missing ingredients which overrode the effects of 12 inches of excellent ballast beneath concrete ties at 24 in. spacing and 132 lb./yd. rail.

TABLE NO. 14
EFFECTIVENESS OF TRACK SURFACE LIFT (MECHANIZED)
SITE NO. 19 MILE 122.3 JOLIETTE SUB.

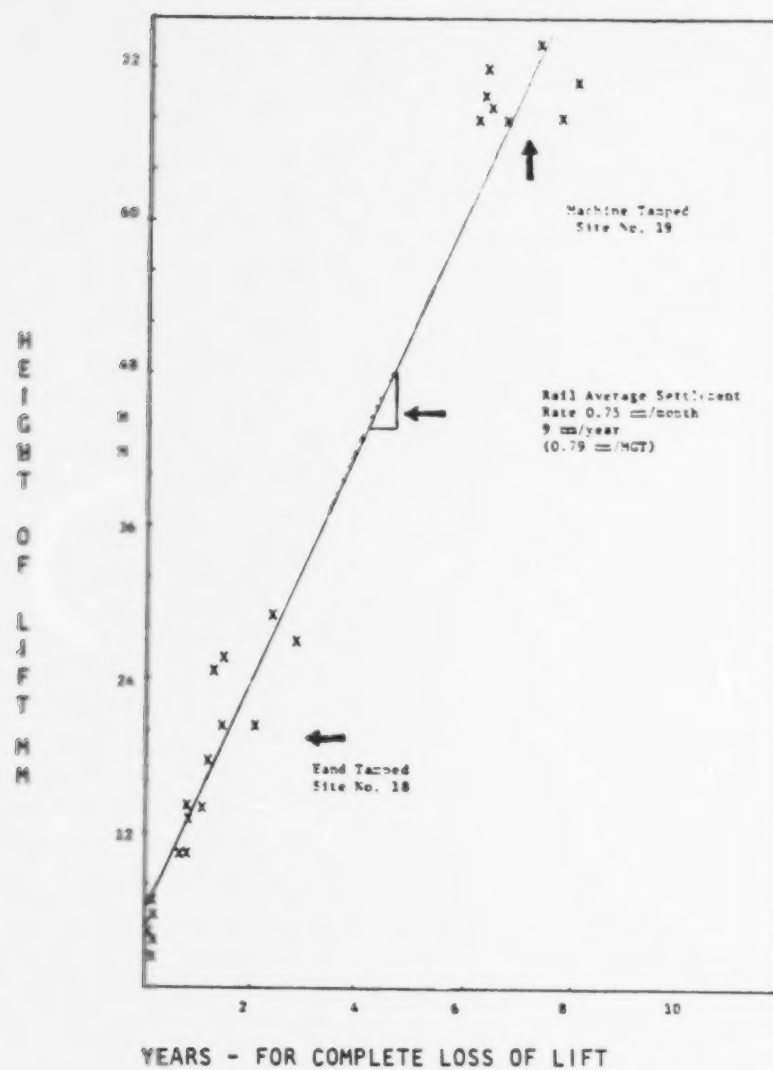
Station No.	Rail Lift mm	*Loss of Lift with Tonnage			Rail Settlement mm/1000 Days	Lift Life Years
		mm	Days	mm/MGT		
1	71	10	69	4.65	21	8.1**
2	70	11	69	5.12	26	6.4
3	69	11	69	5.12	25	6.5
4	72	13	62	6.73	26	6.4
5	68	12	64	6.02	23	6.8
6	74	11	71	4.97	24	7.4
7	68	10	64	5.02	26	6.3
8	68	13	110	3.79	20	7.8
Average	70	11.4	72		24	6.9

Traffic 1981 = 11.369 MGT/yr. $\longrightarrow$ 0.031 MGT/day.

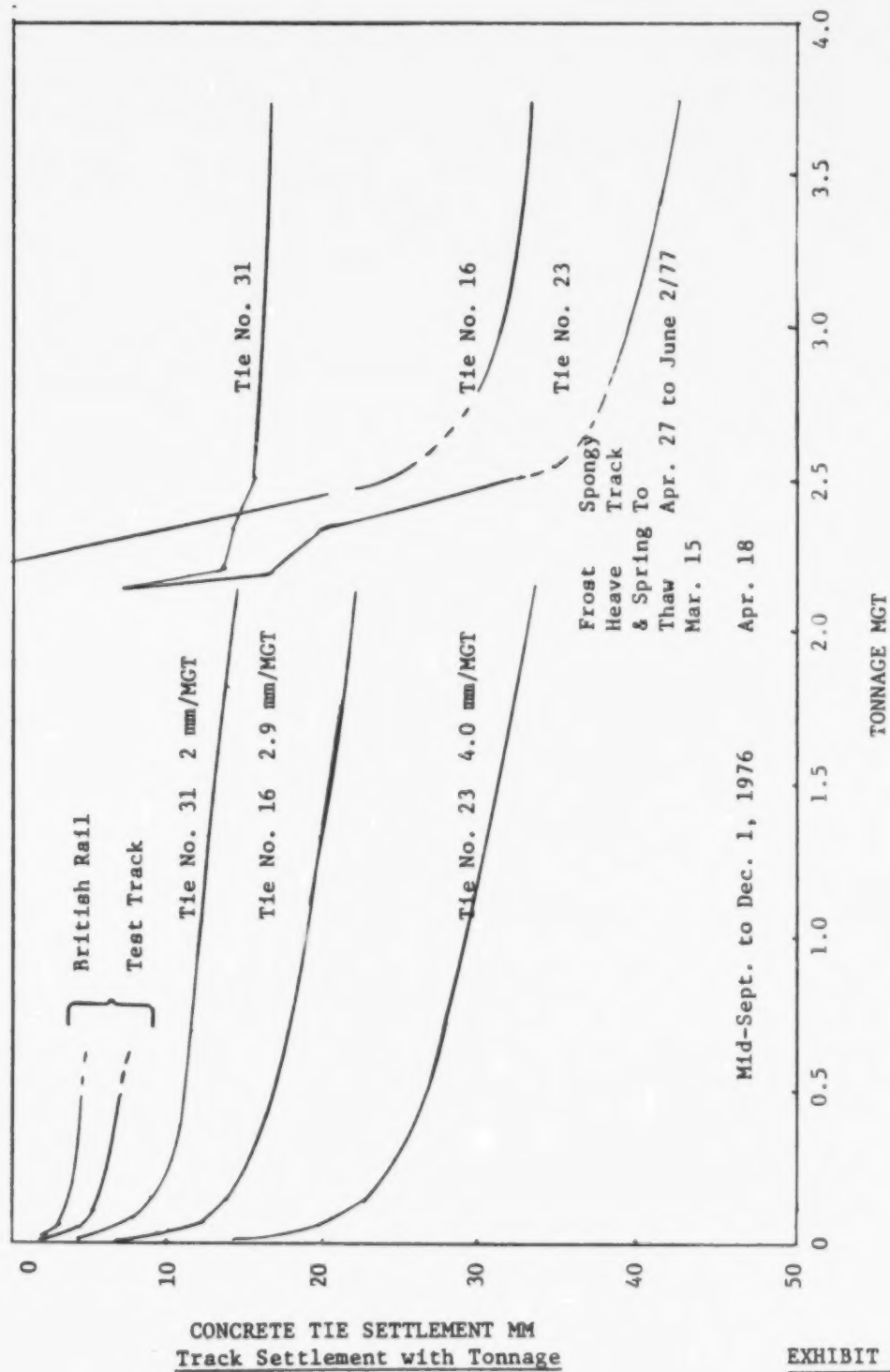
* Rapid loss which occurred following lift.

** Lift Life $\frac{(71 - 10) \text{ mm}}{21 \text{ mm/1000 days}} + 69 \text{ days} = 2974 \text{ days} \div 365 = 8.1 \text{ yrs.}$

1. Average lift was 70 mm (2.75 in.).
2. Loss of 11.4 mm (0.45 in.) or 16% of total lift within 72 days (2.4 months). This corresponds to 2.3 MFT.
3. Effect of lift was lost within 7 years (equivalent to 79 MGT).
4. Average Subgrade Settlement Rate 3.4 mm/1000 days.
Average Rail Settlements Rate 24 mm/1000 days.
Therefore 86% of Settlement occurred within granular layer (-20 in. of granular below base of tie).

LOSS OF TRACK SURFACE LIFT WITH TIME

TEST TRACK AT C N RESEARCH CENTRE VILLE ST. LAURENT, QUE.

SETTLEMENT VS. TONNAGE

CONCRETE TIE SETTLEMENT MM
Track Settlement with Tonnage

EXHIBIT NO. 32

4. TRACK GEOMETRY CAR RECORDS AT TEST SITES

Track surface roughness and cross level Index Numbers as recorded by the CN Track Geometry Car from 1970 to 1982 at the six test site locations are presented in Exhibit No. 33. Track Index Numbers are generated every one-quarter mile of track, and the average of these numbers over 3 to 3-1/2 miles of track have been plotted. The Index Numbers are computed from the slope variations on surface and cross level, with the higher numbers signifying rougher track.

At Site No. 11, Mile 257.8 on the Wainwright Sub., there were gradual improvements in track surface from 1973 to 1977, some deterioration in 1981. There were no wild swings for the worse in Index Numbers, indicating the rate of track surface deterioration was not too rapid.

At Site No. 14, Mile 9.67 on the Edson Sub., the track maintenance level was quite consistent from 1972 to 1976, a very significant improvement was made in 1977, and maintenance levels were consistently good from 1977 to 1981.

At Site No. 16, Mile 123.4 on the Vegreville Sub., Index Numbers were very high and erratic between 1972 and 1977, indicating poor track surface and rapid rates of deterioration. Gradual improvements were made from 1977 to 1981, indicating that the maintenance level had track under control, even though 1981 Index Numbers were twice as high as they were on the Edson and Wainwright Subs.

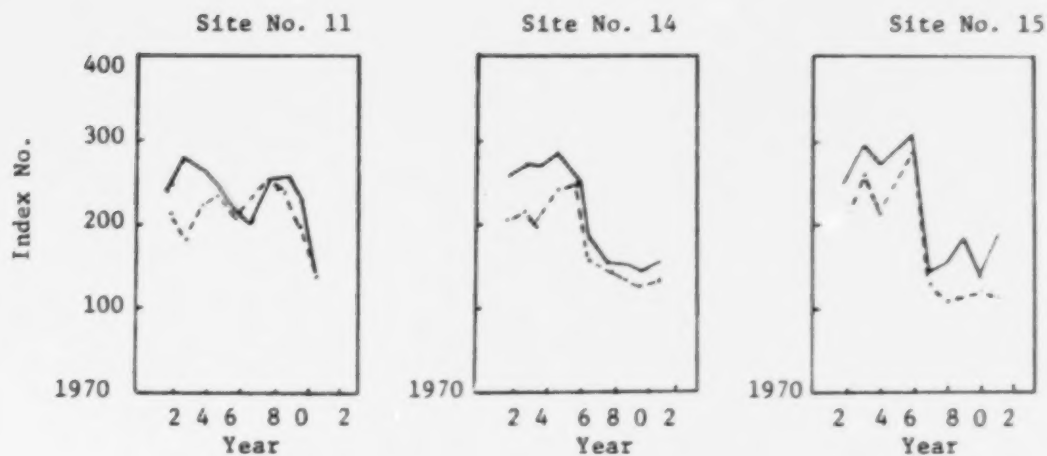
At Sites No. 18 and 19 on the Joliette Sub., the Index Numbers were relatively uniform over the ten-year measurement period, indicating that the maintenance level was sufficient to keep even with the rate of track surface deterioration.

Before the project close-out report is written, an effort will be made to obtain records of track surfacing maintenance at the various test sites.

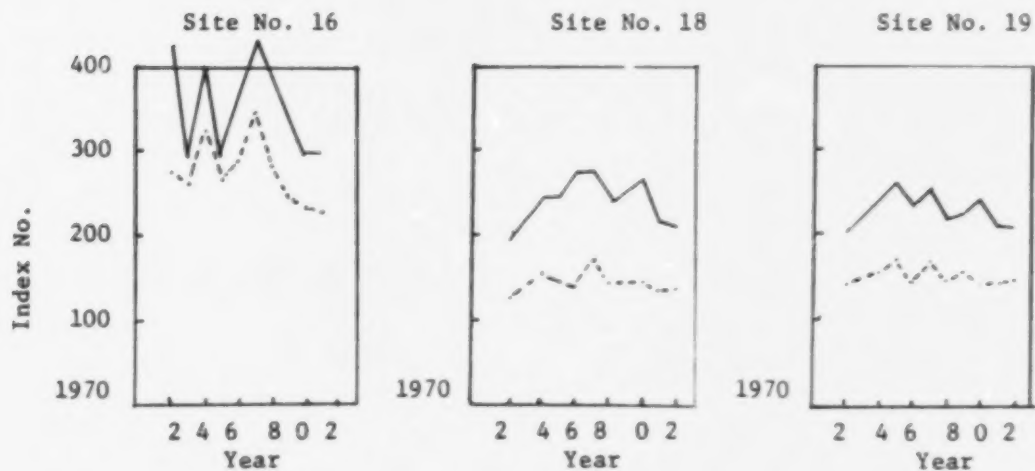
Track Surface Roughness and Cross Level Index Numbers

Index Numbers as Measured by Track Geometry Car were Averaged over 3-3 1/2 Miles Measurements taken primarily in latter part of year.

Site No.	Sub.	Mi.	Mi. Range
11	Wainwright	257.8	256 - 259
14	Edson	9.67	8 - 11
15	Edson	38.7	37 - 40
16	Vegreville	123.4	122 - 125
18	Joliette	118.2	117 - 120.5
19	Joliette	122.3	120.5 - 124



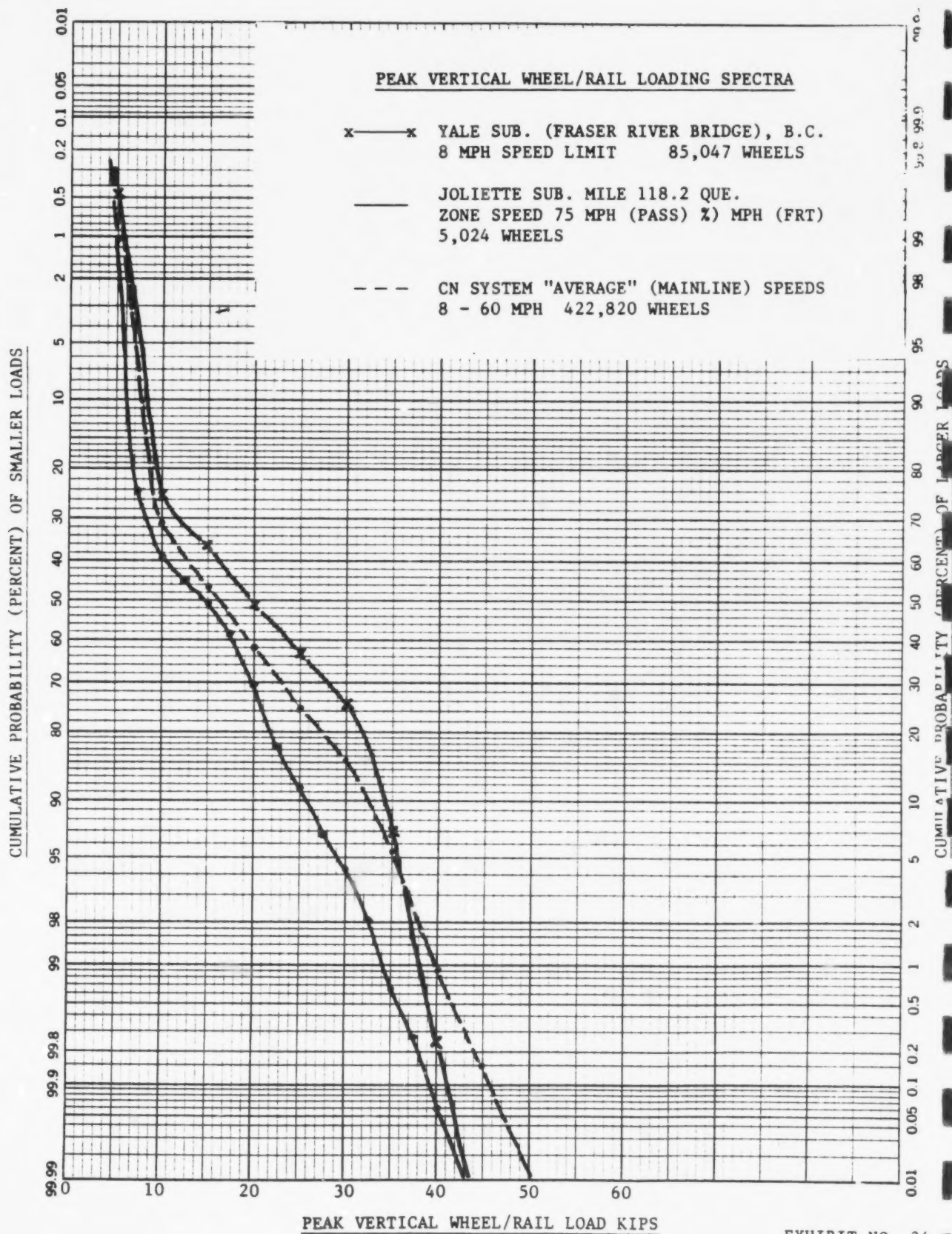
— Surface Roughness Higher Index Numbers
 --- Track Cross Level Signify Rougher Track



5. WHEEL VERTICAL LOADING SPECTRA

Peak vertical wheel/rail loading spectra for the Yale Subdivision (basically representative of the CN Mountain Region mainline track), the Joliette Subdivision, and the CN System "average" are plotted in Exhibit No. 34. It is felt that all wheel loads of less than 20 kips can safely be ignored as far as roadbed design requirements are concerned. This effectively eliminates 50 to 70% of the wheels from consideration. The wheels in the 20 to 30 kip dynamic load range may or may not be important, depending upon the status of the track (mainline or branch) and the relative numbers of heavier loads.

The significance of traffic distribution is clearly illustrated if one looks at wheel loads exceeding 30 kips. On the Mountain Region where there are high concentrations of 100-ton cars in unit train operations, 25% of the wheel loads exceeded 30 kips, while on the Joliette Subdivision there were only 4% of the wheel loads which were above that level. While roadbed elastic deformations are governed by the magnitude of wheel loading, roadbed permanent settlement rates are governed by both the magnitude and frequency of occurrence of loading. Track loading must be characterized in terms of both magnitude and frequency, and these parameters must be related to total yearly tonnage, the parameter for which records are kept. An effort has been made to characterize track loading in terms of these parameters in Appendix D. It is one of the necessary steps which must be taken if we are to arrive at validated granular depth design curves.



6. SUMMARY DISCUSSION

There are four factors which must be correlated in order to establish roadbed design criteria:

1. The properties of the subgrade material.
2. The track loading spectrum (magnitude and frequency of occurrence).
3. The relationship between track loading and roadbed elastic/plastic deformations.
4. An acceptable level of track surface maintenance.

The fourth factor is difficult to define. While everyone would like maintenance-free track, it is impossible to obtain and very expensive to approach. One must weigh the capital construction costs against the future maintenance costs in order to arrive at an optimum design. Exhibit No. 30 (a) has been re-presented here as Exhibit No. 35 in order to re-look at the roadbed settlement rates. At all of the test sites except one, track surface maintenance appeared to be under control (based on the Track Geometry Car records). It would therefore appear that a subgrade settlement rate not exceeding 5 mm/year would be an acceptable limit. This is open to debate and has not yet been related to actual track surfacing costs at the various sites.

The graph of measured subgrade settlement rate (mm/MGT) vs. existing depth of granular, presented previously as Exhibit No. 30 (b), and re-presented here as Exhibit No. 36, has effectively combined the four factors listed above and indicated directly an apparent "maximum effective" depth of granular. For the Lake Edmonton clays, a "maximum effective" depth of granular of 30 in. is indicated. If less granular is used, the settlements per MGT increase dramatically; if more granular is used the settlement rate per MGT is unaltered, indicating minimal return on investment.

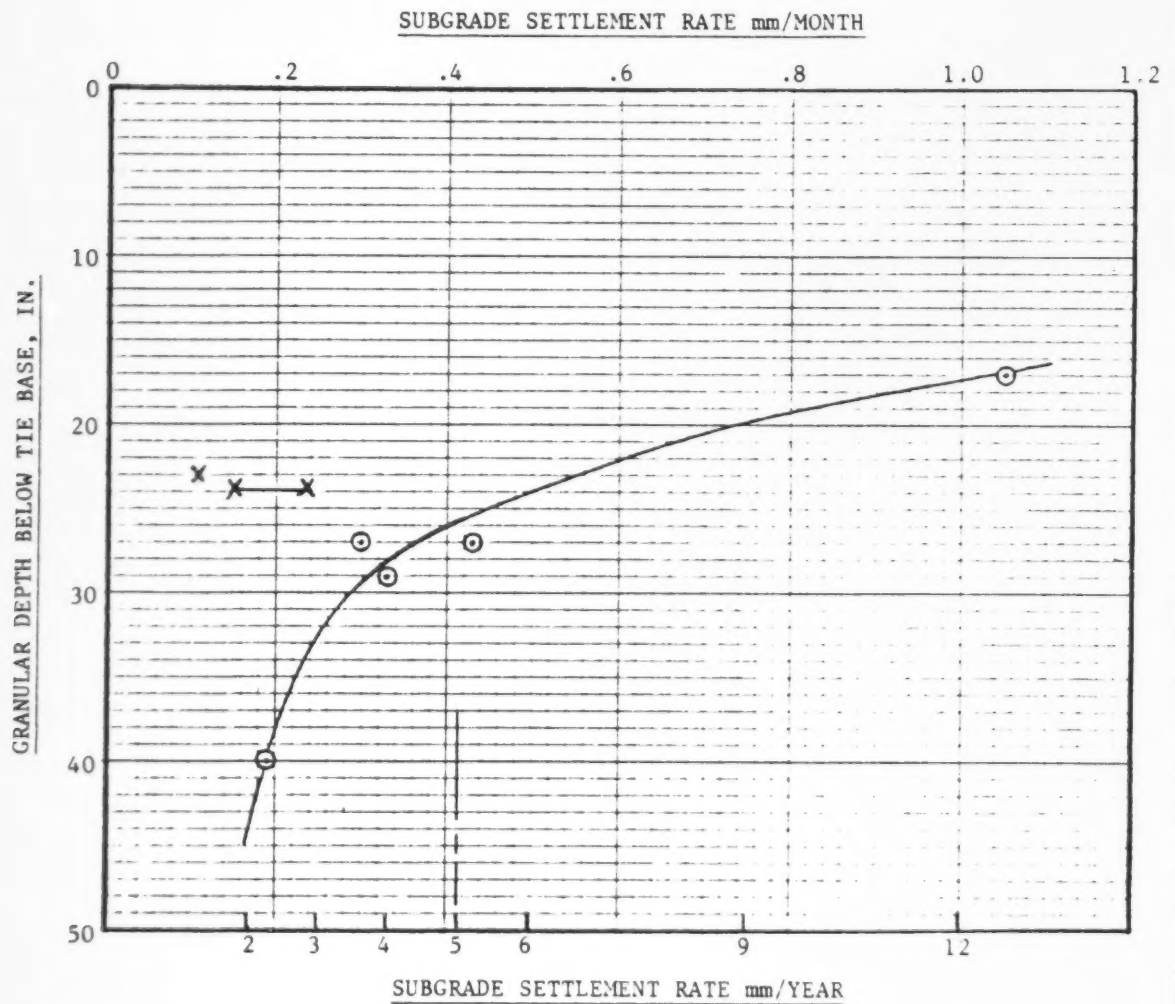
How have the four factors previously listed been effectively combined?

1. The properties of the subgrade need not even be known when taking settlement readings.
2. The measured settlements directly reflect all effects of the track loading spectrum (which need not be measured).
3. The effect of unknown roadbed stress/strain relationships have been directly measured.
4. The plotted curve showed a sharp break point, directly indicating "maximum effective" depth of granular.

No theories were required, no soils tests were required, no characterization of traffic was required, no elaborate hardware or instrumentation was required (simple settlement plates and rod and level). If this study

SUBGRADE SETTLEMENT RATE ON TIME BASIS

(Includes Various Traffic Densities)

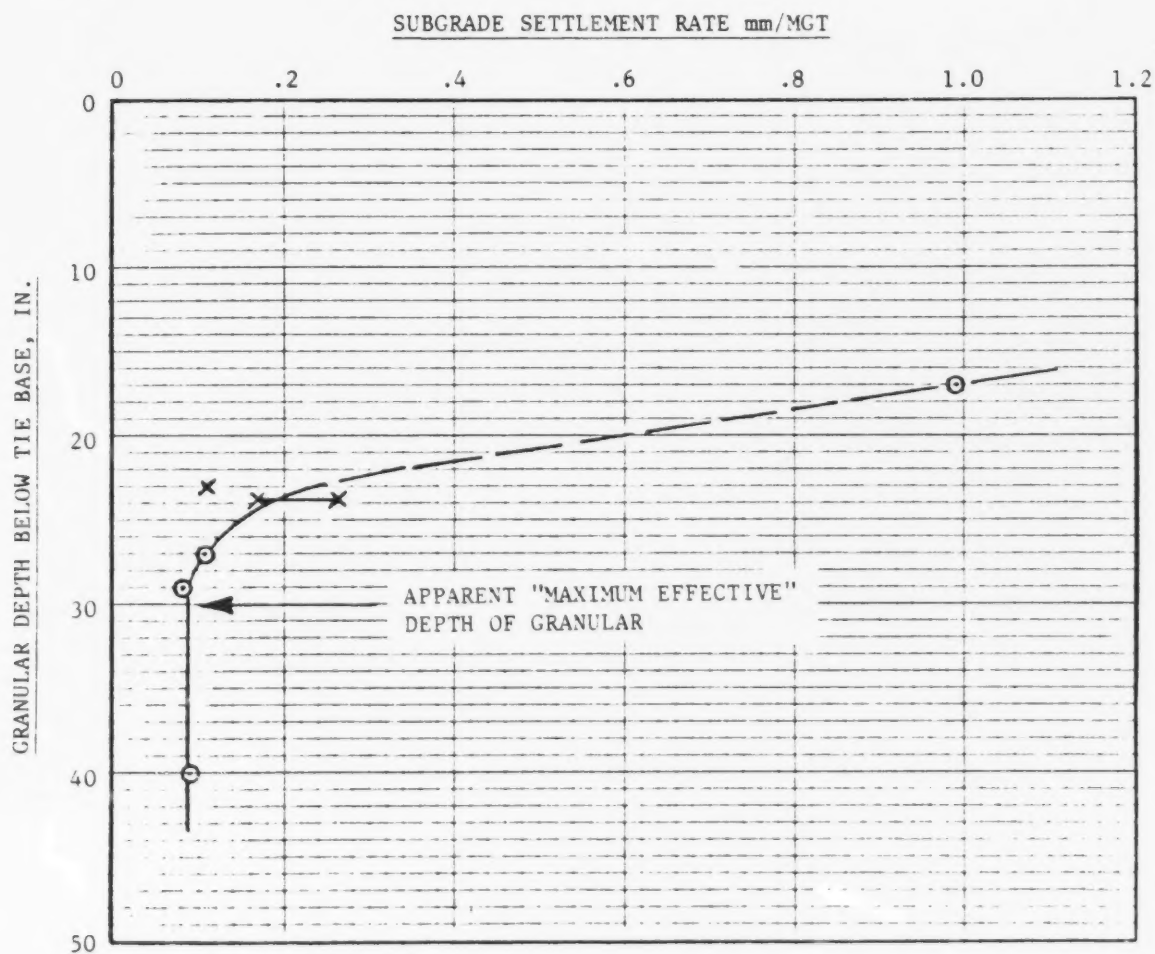


⊙ Lake Edmonton Clay

x Silt (Joliet Sub.)

SUBGRADE SETTLEMENT RATE ON TRAFFIC BASIS

(Includes Various Time Frames)



⊙ Lake Edmonton Clay

x Silt (Joliet Sub.)

has provided no other useful information, it has clearly shown how depths of granular required for future traffic can be determined from simple measurements on existing lines.

It will be noted that the only external information required to plot Exhibit No. 36 was the traffic density, and this information is recorded yearly for every Subdivision in the System.

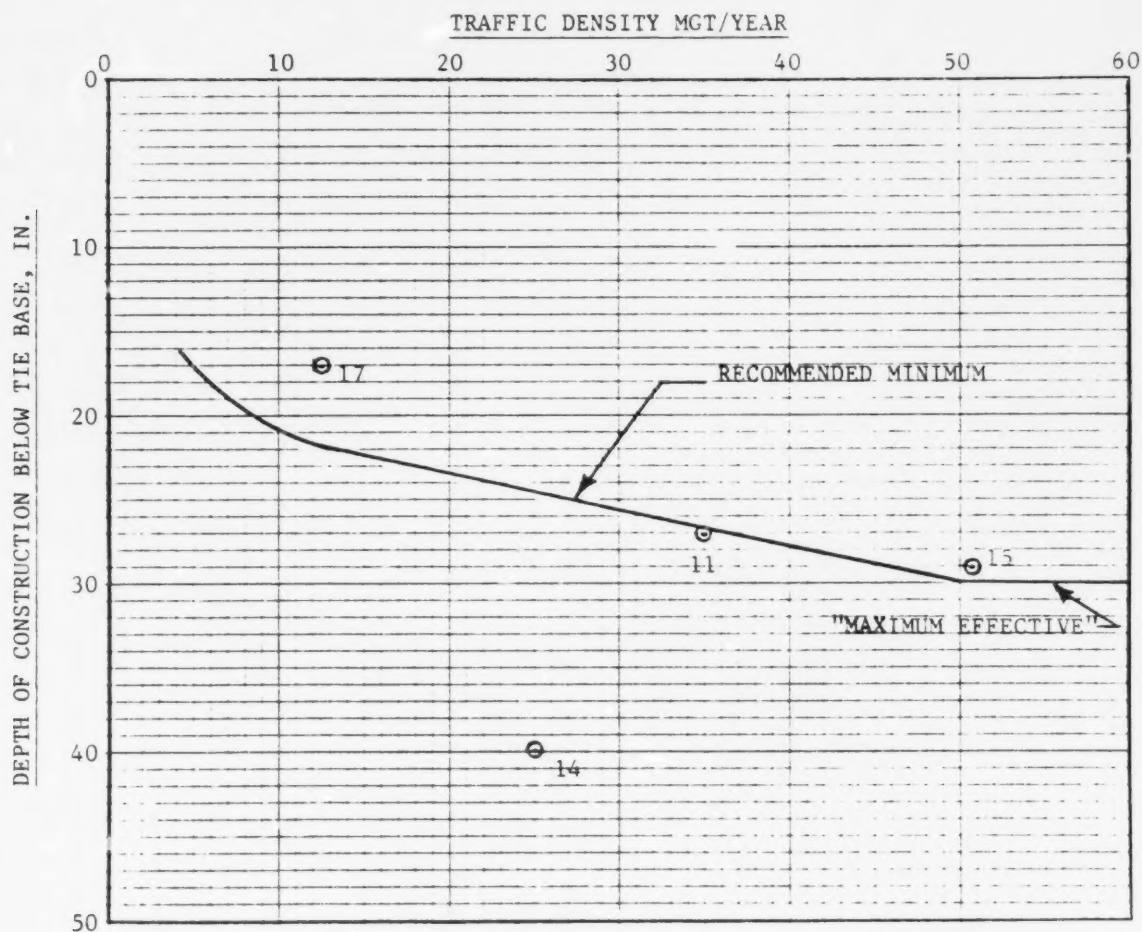
Based upon Exhibit No. 36, it is possible to specify the depth of construction required for different traffic densities. A "Guideline for Depth of Construction on Lake Edmonton Clay" has been presented in Exhibit No. 37. The recommended construction depth is based upon subgrade settlement not exceeding 5 mm/year and traffic densities with 20% of the axle loads greater than 30 tons. Existing construction depths and traffic densities at the four test sites have been shown. At Test Sites 11 and 15, existing construction depths are exactly as recommended for the traffic densities. At Test Site 14, there is 16 in. excessive granular depth (traffic was halved in 1979 due to double tracking). Track surface maintenance intervals at this site would basically depend upon settlements within the granular layer and not settlements of subgrade. Even if traffic were to double or triple at this site, it is still considered that there is 10 in. more granular depth than required. At Site No. 17, there is insufficient granular depth. The present depth of 17 in. would be sufficient for 5 MGT of traffic, but for the present 12 MGT, 22 in. of granular is required if track maintenance levels are to be held at acceptable levels.

While the above approach is an oversimplification, it has been very effective. For every track and subgrade soil type, there should be a "maximum effective" depth of granular, beyond which the law of diminishing returns will govern. There will not be good return on investment. There is an "optimum" depth of construction which varies with traffic density. On the Lake Edmonton clay subgrade, one might say that "optimum" construction depths are 17 in. for 5 MGT traffic and 30 in. for 50 MGT traffic. Maintenance requirement should be within similar and acceptable limits on both tracks.

A knowledge of the relationship between "optimum" construction depth and traffic density is extremely important for scheduling ballast requirements for lines with projected traffic increases. Track surface lifts could be pre-scheduled in order to hold maintenance within acceptable limits. For new track construction, where traffic increases were projected over a period of years, appropriate construction depths could be selected. For example, it may not be economical to construct with 30 in. granular if traffic were projected to be only 25 MGT after 25 years. This is a matter of return on investment and will not be dealt with in this Report.

For existing lines, there is a problem in direct application of the "optimum" depth guideline. If one were faced with upgrading hundreds of miles of track (possibly constructed 50 or 100 years ago), the existing depths of granular and their variations along the track would be unknown. Unless these were determined by digging hundreds of holes in the roadbed, it would not be possible to specify the additional granular depth requirement in order to reach optimum.

PRELIMINARY
GUIDELINE FOR DEPTH OF CONSTRUCTION
ON LAKE EDMONTON CLAY
FOR COMPARABLE TRACK SURFACE MAINTENANCE



- Based on acceptable subgrade settlement rate of 5 mm/year.
- Traffic density with ~ 20% axle loads > 30 tons, speeds less than 50 mph.

⊙ Existing depth of granular and traffic density at Site No.

There are two possible ways to get around this problem:

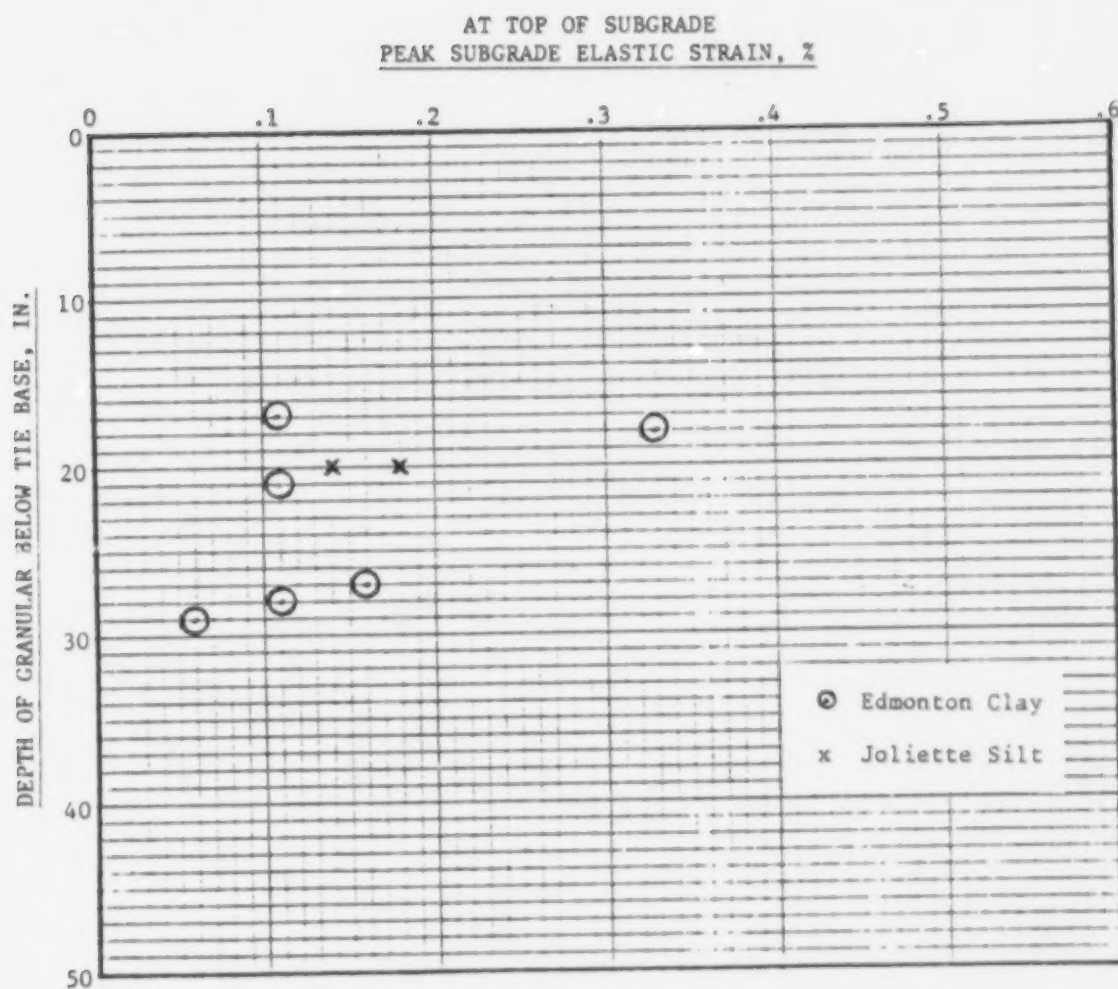
1. Development of equipment which could very quickly measure existing granular depth (drill or punch), in order that several measurements per mile could be taken.
2. Development of relationships between track elastic deflections and granular depth requirements.

If roadbed elastic deflections could be measured from a moving vehicle, they could be recorded in a very short time frame. If correlations between roadbed elastic behaviour had previously been made with roadbed settlement rates (through a research program), then a very practical means of determining track upgrading requirements would have been established.

We will now re-look at the information in this Report and the two previous Progress Reports^(1,2), and establish some of the correlations between roadbed elastic behaviour and settlement rate. At the "maximum effective" depth of granular of 30 in. as established for Lake Edmonton clays by the settlement graph, it has been determined that:

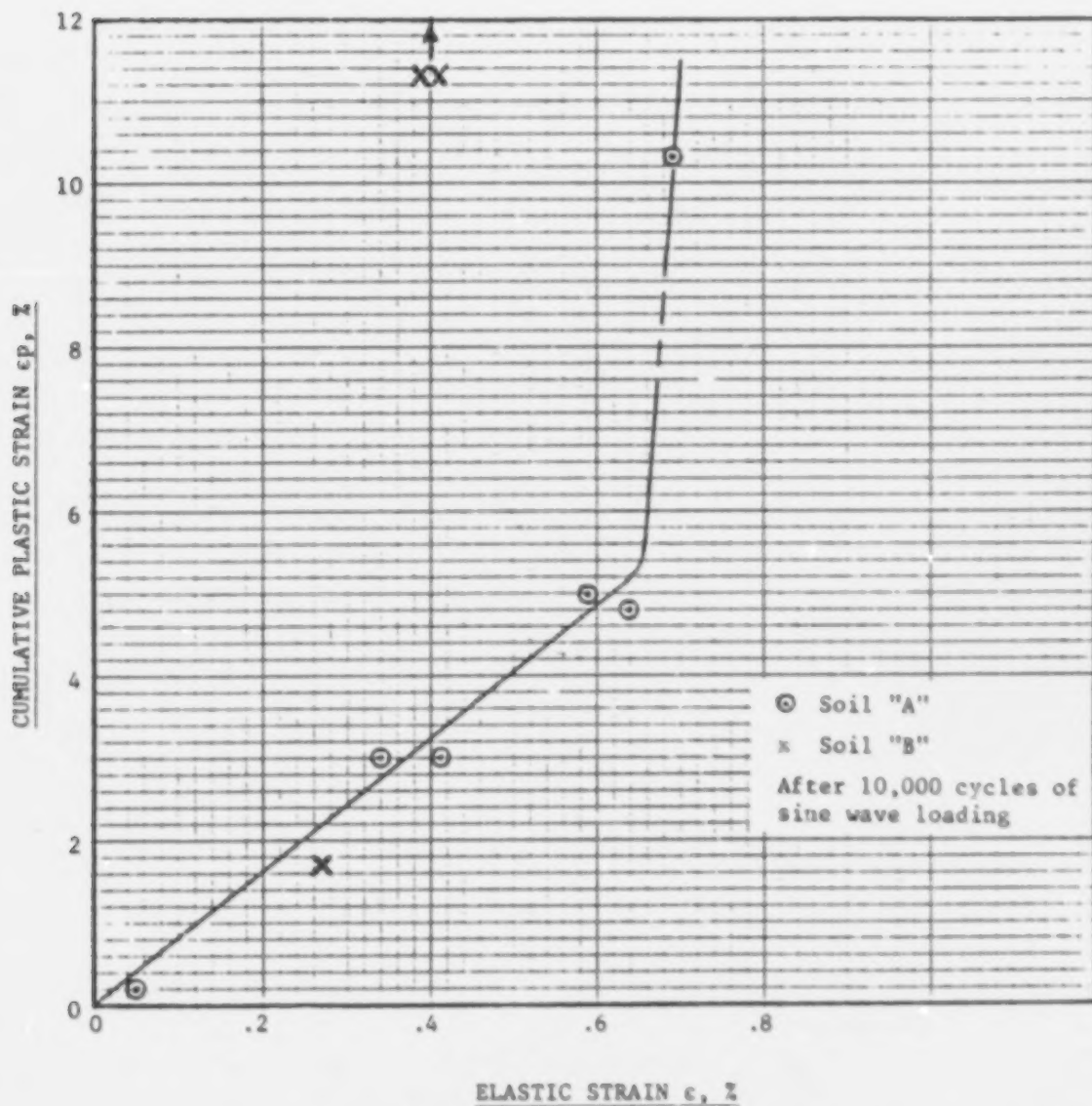
1. The ratio of subgrade elastic deflection to rail/tie deflection should not exceed 0.33 (slack should not be included in rail/tie deflection).
2. The elastic deflection at top of subgrade should not exceed 0.0015 in./kip of tieplate loading.
3. The rail/tie elastic deflection should not exceed 0.0045 in./kip of tieplate loading.
4. Elastic deflection at top of subgrade should not exceed 0.04 in. and elastic deflection of rail/tie should not exceed 0.125 in.
5. From the peak elastic deflection curve and the normalized elastic deflection curve, a tieplate load of 28 kips has been established as nominal maximum.
6. The peak elastic strains in the top surface layer of subgrade (established from slopes of roadbed deflection vs. depth curves) have been plotted against depth of granular in Exhibit No. 38. Subgrade elastic strains should not exceed roughly 0.1% if excessive subgrade settlements are to be avoided.
7. The relationship between soil plastic and elastic strains, as determined by McGill University following 10,000 cycles of sine wave loading in dynamic triaxial tests, is shown in Exhibit No. 39. This graph indicates a soil elastic "threshold" strain of 0.65%, the same as that established by British Rail for London clay. The maximum permissible soil strain of 0.1% in the roadbed is far less than the soil "threshold" strain. A future Report will document correlations between laboratory and field measurements.

SOIL STRAINS AT TOP OF SUBGRADE



Site No.	Soil Type	Top of Subgrade, In.	Elastic Soil Strain, %
3	Edm.	28	0.11
4	Edm.	21	0.11
9	Edm.	18	0.33
15	Edm.	29	0.06
16	Edm.	17	0.11
17	Edm.	27	0.16
18	Jol.	20	0.18 (1980)
		20	0.14 (1981)

McGILL DYNAMIC TRIAXIAL TEST DATA ON LAKE EDMONTON CLAY



Soil	Test No.	Dev. Stress, psi	c_p , %	e , %
A	4	3.2	0.2	.045
A	6	5.6	3.0	.34
A	10	6.0	3.0	.41
A	5	6.4	4.8	.64
A	7	7.2	10.3	.69
A	16	6.6	5.0	.59
B	12	6.6	>14	~.4
B	14	6.6	1.7	.27
		7.2	>14	~.4

8. It is clear that optimum depth of construction depends upon traffic density (and loading distribution within that density). What may have been optimum depth 50 or 100 years ago is insufficient today. For any rail line, the subgrade settlement rate should not exceed 5 mm per year, and optimum depth of construction would then be determined for each line depending upon soil type and present or future traffic density.
9. Very few tracks are in danger of catastrophic failure if "over-loading" (both magnitude and frequency) occurs. Maintenance costs simply rise. The overriding design criteria for track must therefore be based upon maintenance. For acceptable levels of track maintenance, subgrade settlement rates should not exceed 5 mm/year.

As subgrade settlements were only measured at two sites on the Joliet Subdivision, it has not been possible to determine an optimum depth design curve for the silt subgrade. It has been possible to draw several other conclusions.

1. On average, subgrade settlement rates in mm/MGT for the silt were the same as and/or lower than for the Edmonton clay for similar depths of granular.
2. Based on the criteria of subgrade settlement rate not exceeding 5 mm/year, it would appear that roughly 24 in. depth of granular would be a minimum for the present 11 MGT/year on the Joliet track. Settlement rates at the 20 measurement stations ranged from 0.5 to 5 mm/year where depths of granular ranged from 19 to over 30 in. (no clear correlation between settlement and depth). There were several ties pumping mud in the test site area, a clear indicator that in localized spots there was insufficient granular.
3. If traffic density were suddenly to increase to 50 MGT/year on this line, subgrade settlements would range from 2 to 22 mm/year. It appears that 3 in. or more of granular would have to be added to hold maintenance levels. Adding granular on top of mud-pumping ties will not solve the problem. The subgrade has worked its way up to tie level and must be dug out and replaced with granular.
4. Top of subgrade elastic deflections were roughly 0.002 in./kip of tieplate load.
5. Peak elastic deflection at top of subgrade (taken as 20 in. from base of tie) was 0.04 in. (identical to the level not to be exceeded in the Edmonton clay).
6. From the normalized data, it would appear that a tieplate load of 20 kips would represent the nominal maximum at this site. On average, tieplate loads ranged from 21 to 50% of wheel loads, and dynamic wheel loads ranged from less than 5 to 42 kips. A decision was made halfway through this study that wheel loading should be taken as the reference because it was easier to measure and categorize, and tieplate loading was just another (and worse) variable.

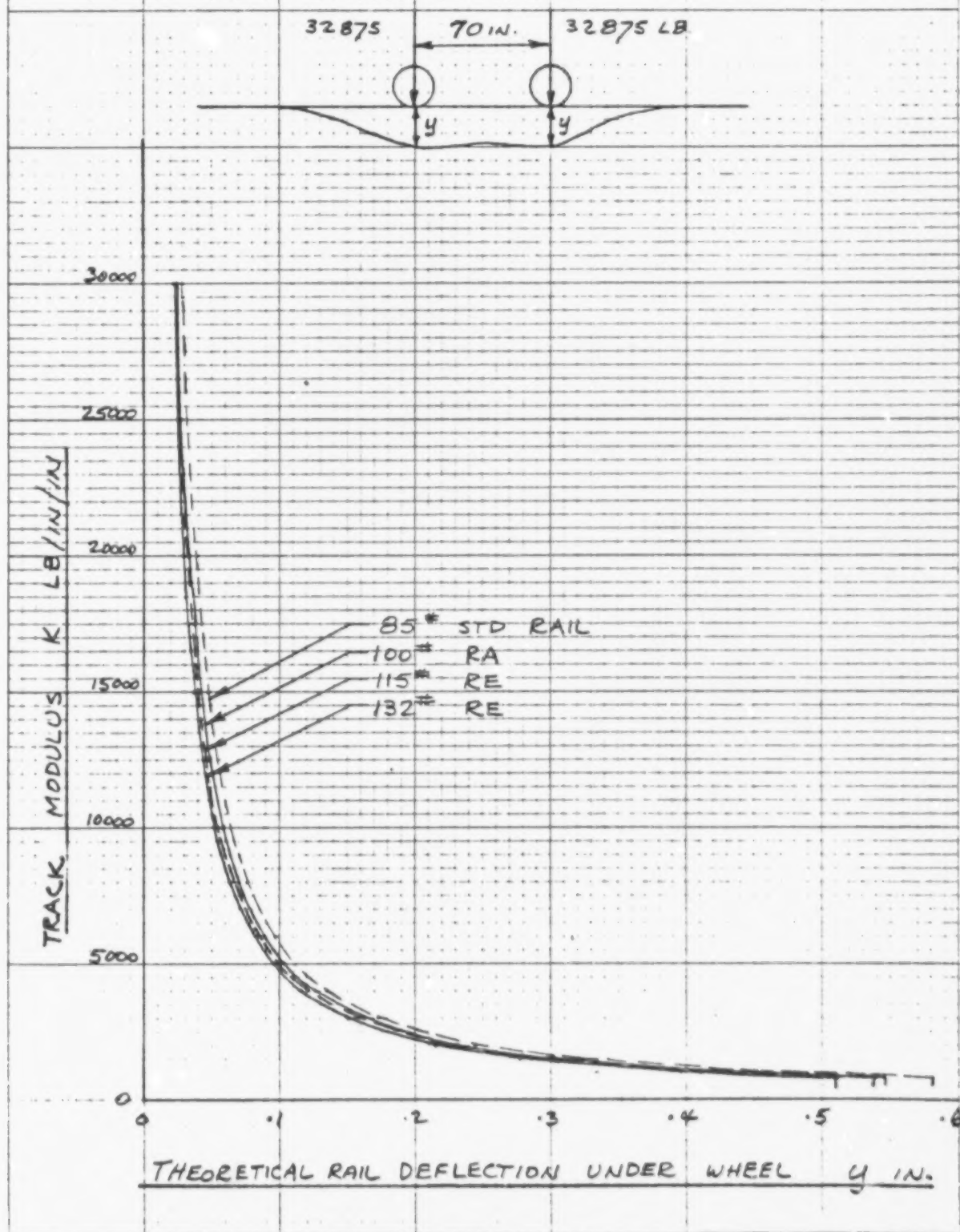
7. Top of subgrade elastic strains ranged as high as 0.18% at 20 in. depth. It appears that maximum permissible elastic strains in the silt should be somewhere in the range 0.1 to 0.15%.
8. Roadbed deflections were measured between two ties, one which was solid and the other which was old and spongy. Solid tie deflections ranged up to 0.09 in., spongy tie deflections ranged up to 0.34 in. (slack eliminated). Average tie deflection ranged up to 0.21 in.
9. Of the total track settlement measured at the Joliette sites, 15 to 25% occurred within the subgrade and 75 to 85% occurred within the granular layer. Subballast was pit-run gravel and older ballast was crushed rock but with considerable fines. Surface ballast, applied in more recent years, was good by today's standards. This clearly points out that the granular layer itself can play a large role in track settlement rates. Unfortunately, history cannot be rolled back, and upgrading of old lines constructed with poor quality ballast and subballast can pose more problems than construction of new lines.

It has been stated that, for tracks constructed over Edmonton clay, rail/tie elastic deflections should not exceed 0.125 in. if the line is to carry 50 MGT per year with acceptable maintenance. Exhibit No. 38 presents a plot of track modulus vs. rail deflection under the wheel of a fully loaded 100-ton freight car. The curves have been derived from the classic beam-on-elastic-foundation theory. A deflection of 0.125 in. corresponds to a track modulus of 3500 to 4500 lb./in./in., depending upon the weight of rail. For 132 lb./yd. rail, a track modulus of 3500 lb./in./in. corresponds to a vertical track stiffness of 0.25 million lb./in. If track modulus or vertical stiffness or rail deflection could be continuously measured from a moving 100-ton vehicle, it may be possible to predict roadbed granular depth requirements for projected traffic on existing rail lines. This, of course, is dependent upon solid data correlating roadbed elastic deflections with subgrade settlement rates.

Not all of the possible cross-correlations of data have been made, but all the information from the CN field measurement program has now been presented. It remains for the field measurement data to be correlated with the laboratory soils test data, and McGill University Geotechnical Research Centre will be doing that over the course of the next six months. In addition, more work is required on definition of the track loading spectra.

The authors would be remiss if attention were not paid to work conducted by others. Appendix E contains a brief literature review of some published information. It is not intended as a comprehensive review, but rather more to highlight some of the factors involved.

THEORETICAL RAIL DEFLECTION UNDER WHEEL
OF FULLY LOADED 100^T CAR.



7. CONCLUSIONS

It is premature to draw final conclusions to this study when the field measurement data still has to be correlated with the laboratory soils test data. This will be done within the next six months. Regardless of whether the information can be correlated, and irrespective of the soil properties, several strong conclusions can be drawn concerning the field measurement program.

1. The apparent "optimum" depth of granular for track constructed over Lake Edmonton clay is:

30 in. for 50 MGT/yr. traffic density
24 in. for 25 MGT/yr.
20 in. for 8 MGT/yr.
17 in. for 5 MGT/yr.

The construction depths with corresponding traffic should all give roughly 5 mm of subgrade settlement per year and all tracks should have comparable and acceptable surface maintenance requirements (assuming all traffic has similar existing average loading spectra). None of the above would apply if concentrations of unit trains with 100-ton cars were suddenly introduced on the territories.

2. In order to provide correlation between traffic density, type of subgrade, depth of construction, and track maintenance requirement, the most important parameters to measure in the field are rail and subgrade settlement rates.
3. Subgrade settlement rates, elastic deflections, and elastic strains can all be measured with relatively simple field instrumentation.
4. Measurement of track modulus (elastic rail deflection) from a moving 100-ton vehicle may provide a means to predict ballast requirements for existing lines facing significant traffic increases.
5. "Optimum" depths of construction for the Joliette silt subgrade can only be estimated because of insufficient test site measurements.

A great many more conclusions relating field measurements with laboratory soils test results will be drawn in the project close-out Report to be published in early 1984.

8. RECOMMENDATIONS

On account of the current North American recession and very heavy work-loads within CN Rail Research, this project will be terminated within the next few months. It is felt that significant findings have been made, but it is also very clear that much more work could be done. It is perhaps a good thing to have a breathing period in which to assess the usefulness of the information. If future work is desired, the experience gained through work on this project should prove invaluable in establishing any new research programs.

It is now recommended that:

1. The "optimum" depths of granular as indicated in the guideline for various traffic densities in the Edmonton area should be compared with actual track maintenance experience at a number of locations. This should be done to confirm the validity of the guideline and/or determine scatter bands which should be applied.
2. On Site No. 18 on the Joliette Subdivision, rail and subgrade settlement rates should be measured during 1983 in order to assess the effects of a track surface lift scheduled for May. Maintenance had been deferred at this location in order to protect buried instrumentation. It should now be possible to determine the effectiveness of the surface lift (increase in granular depth) in relation to maintenance requirement.
3. All of the field measurement data must be correlated with all of the laboratory soils test data (accumulated by McGill University Geotechnical Research Centre) in order to develop empirical or theoretical relationships useful for predicting granular depth requirements.
4. Roadbed analytical models must be molded to conform to measured field data. Field measurement data in statistical strength must come first - history has proven it virtually useless to attempt to validate preconceived theories through sporadic and usually inadequate field measurements. Empirical relationships will suffice if they are supported with field measurements.

BIBLIOGRAPHY

The listed reports are those which have been published in connection with the CN Rail Research Subballast Thickness Study.

1. Dalton, C.J., Scott, J.F., "Predicting Requirements for Subballast Thickness, Progress Report No. 1, Measurements of Track Loads and Deflections, Edmonton, Alberta, May 1979". CN Rail Research Report No. 158, published February, 1980. Transport Canada, Transportation Development Centre, Report No. TP 2584.
2. Dalton, C.J., Scott, J.F., "Predicting Requirements for Subballast Thickness, Progress Report No. 2, Measurement of Track Loads and Deflections, Edmonton, Alberta, June 1980, L'Assomption, Quebec, August 1980". CN Rail Research Report No. 166, published April, 1981. Transport Canada, Transportation Development Centre, Report No. TP 2987.
3. Sheeran, D.E., Geotechnical Research Centre, McGill University, Montreal, Quebec. "Dynamic Triaxial Testing Program, Present Status." Report prepared for CN Rail Research Centre, July 1979.
4. Sheeran, D.E., Elmallouk, H.A., Fattah, E.A., Geotechnical Research Centre, McGill University, Montreal, Quebec. "Prediction of Long Term Track Subgrade Settlement Due to Cyclic Wheel Loading", November 1980. CN Rail Research Report No. 164. Transport Canada, Transportation Development Centre, Report No. TP 2648.
5. Sheeran, D.E., Jayaraman, R., Geotechnical Research Centre, McGill University, Montreal, Quebec. "An Interim Report for the Design of Ballast Thickness Under Adverse Conditions", September, 1981. CN Rail Research Report No. 177. Transport Canada, Transportation Development Centre, Report No. TP 3766E.

Reference material published by others is listed in Appendix E, Subballast Thickness Study, Literature Review.

APPENDIX A

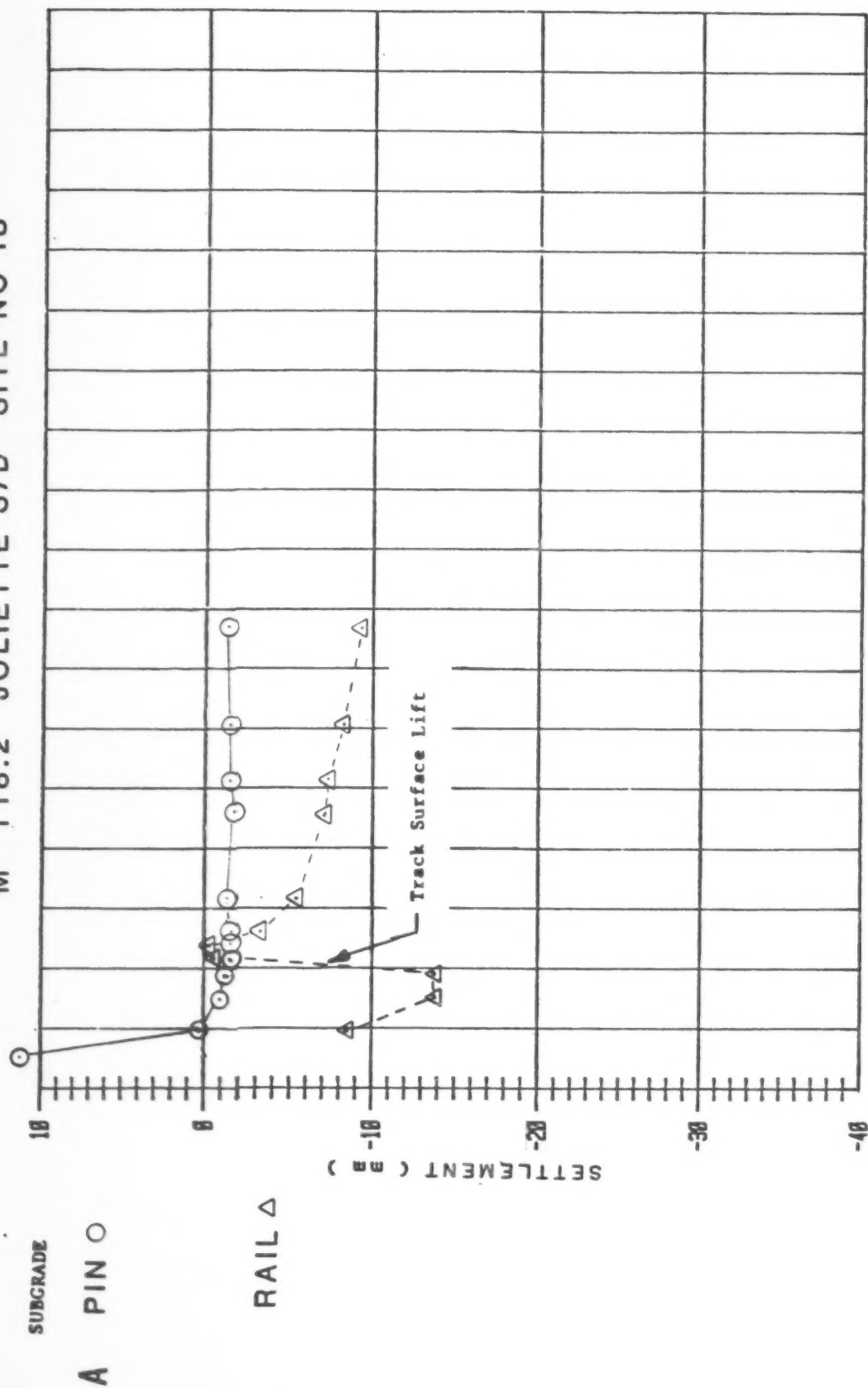
Subgrade and Rail Settlements Site No. 18, Mile 118.2 Joliet
Subdivision Period 1980 - 1982.

Note: All rail and subgrade elevations were taken with precise level with reading accuracy of ± 0.01 mm.

Rail elevations were measured on rail head. Subgrade elevations were measured on top of 1/2 in. dia. rod (pin) which was welded to 6 in. square steel plate resting on top of the subgrade. The rod was enclosed in a plastic tube to minimize frictional effects from the granular layer.

SETTLEMENT GAUGE READINGS

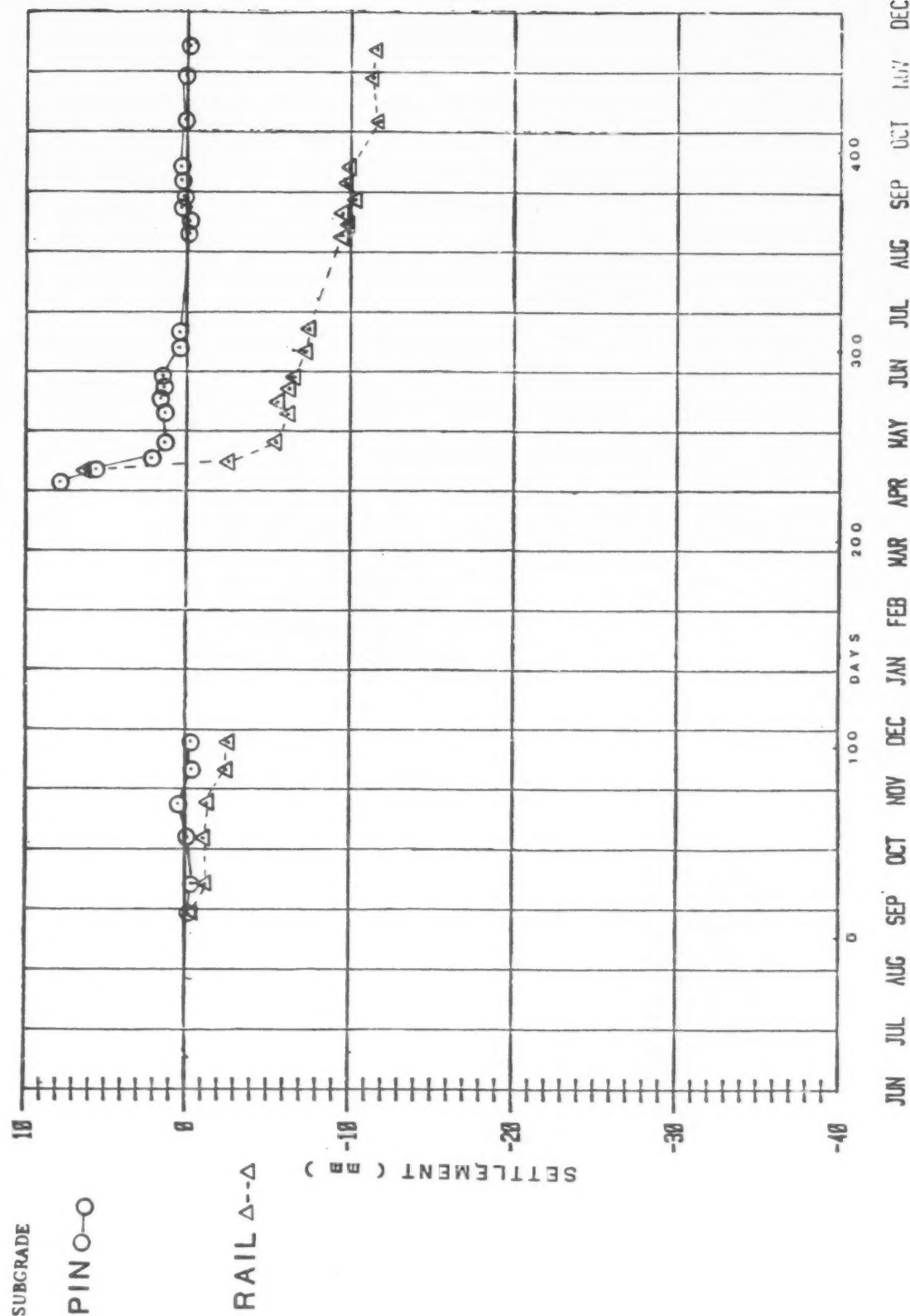
M 118.2 JOLIETTE S/D SITE NO 18



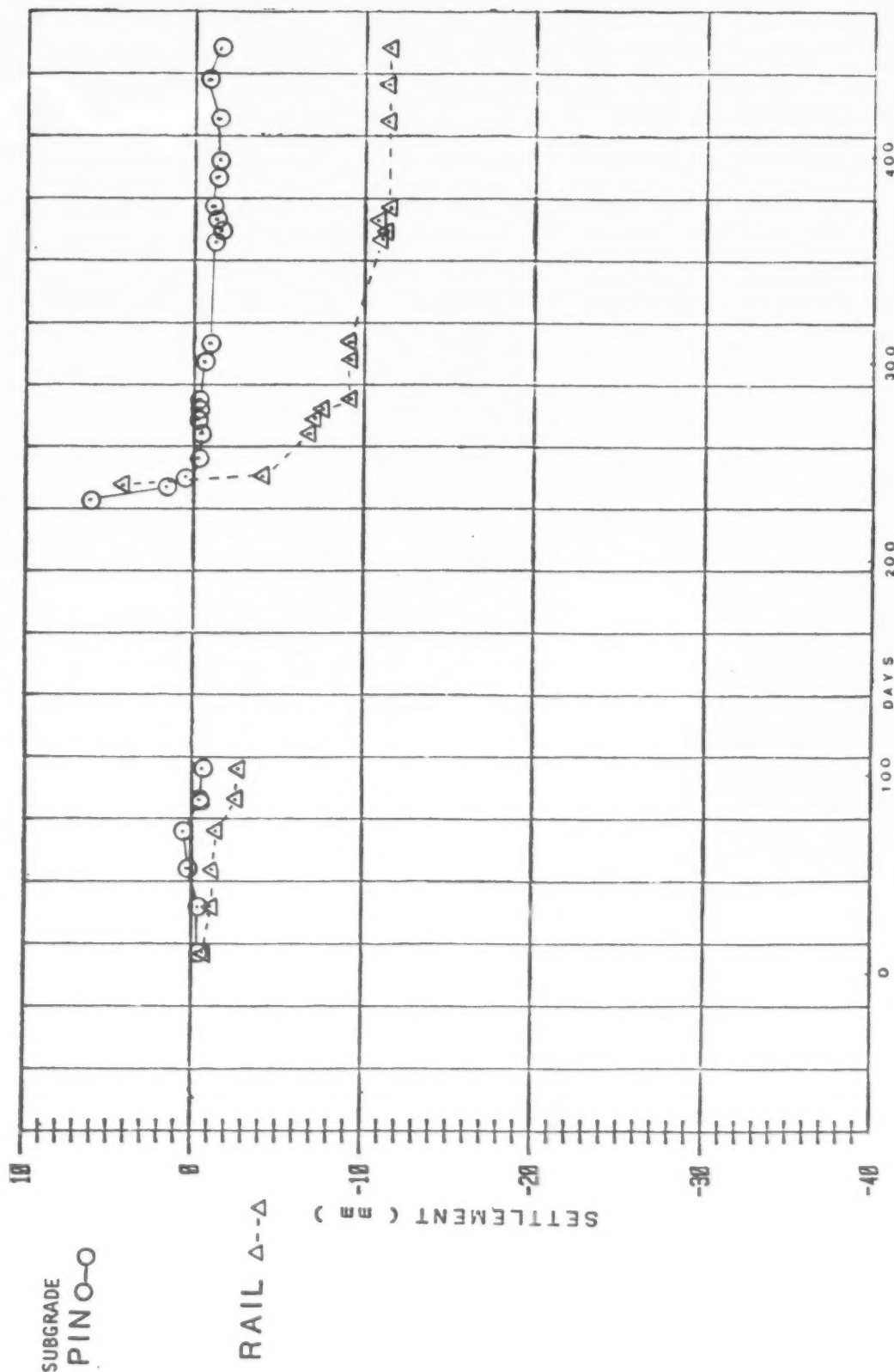
APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-A



SETTLEMENT GAUGE READINGS M 118.2 JOLIETTE S/D SITE NO 18-B

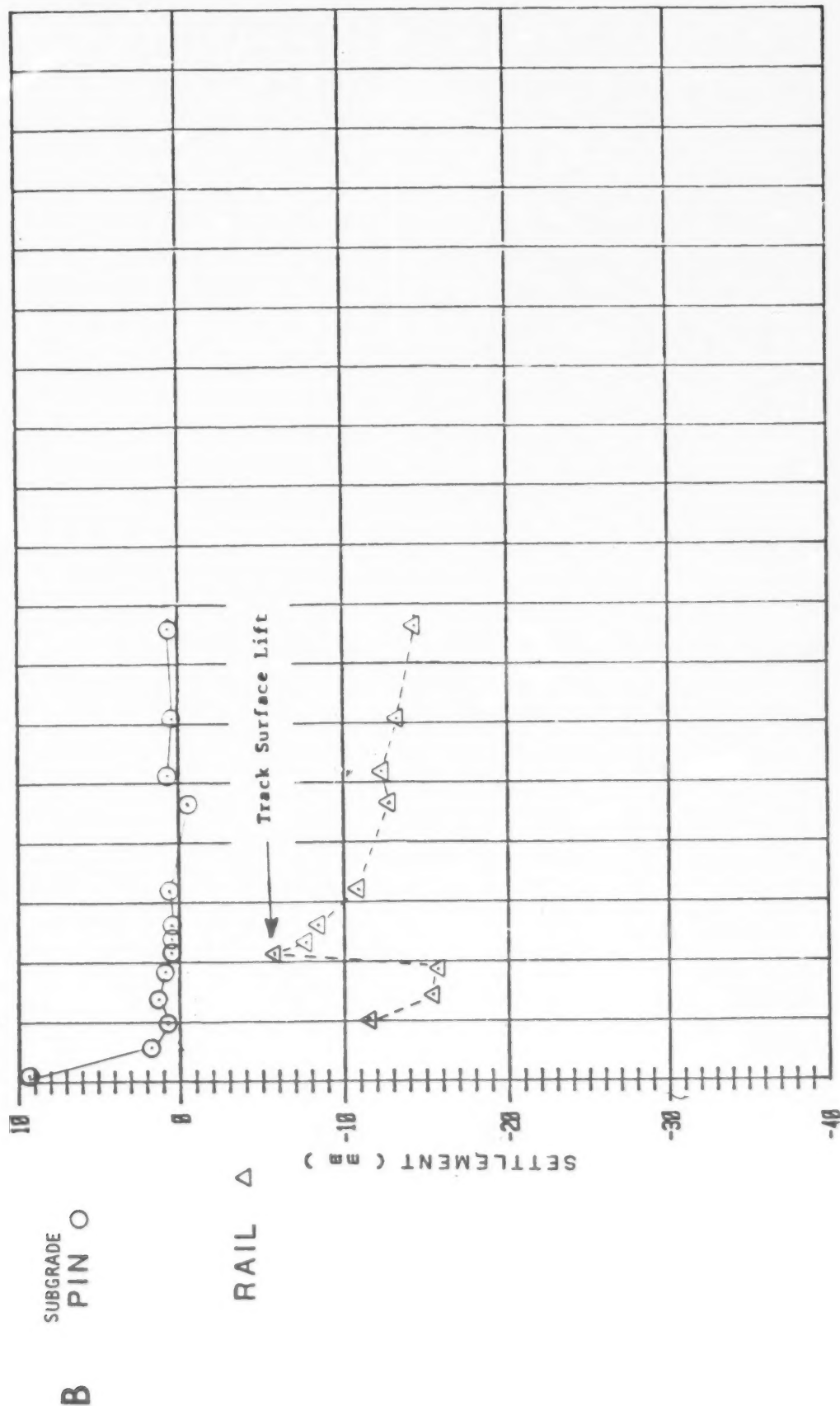


JUN JUL AUG SEP OCT NOV DEC

MONT 1980 1981

SETTLEMENT GAUGE READINGS

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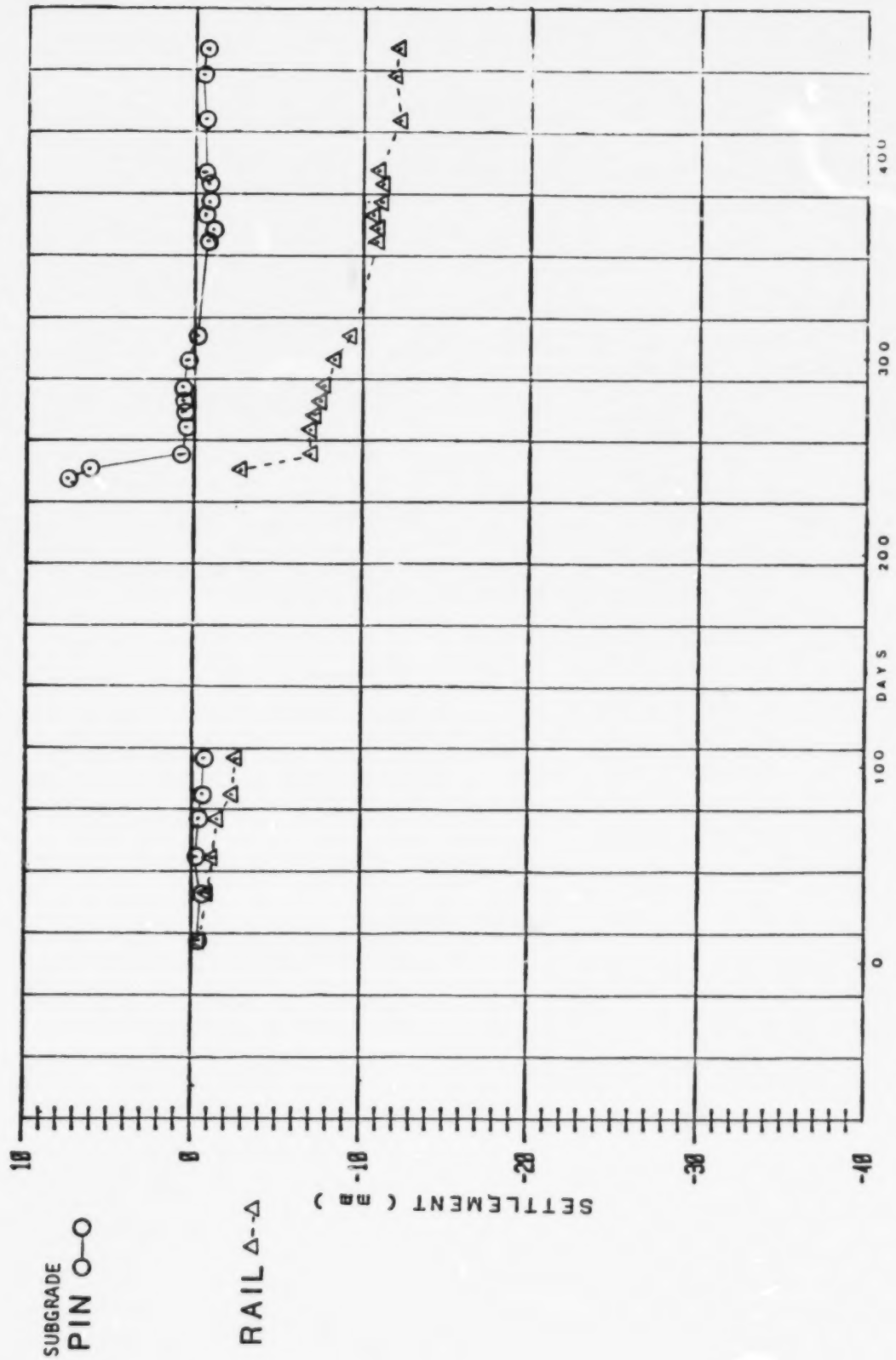


APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

1982 MONTHS

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-C

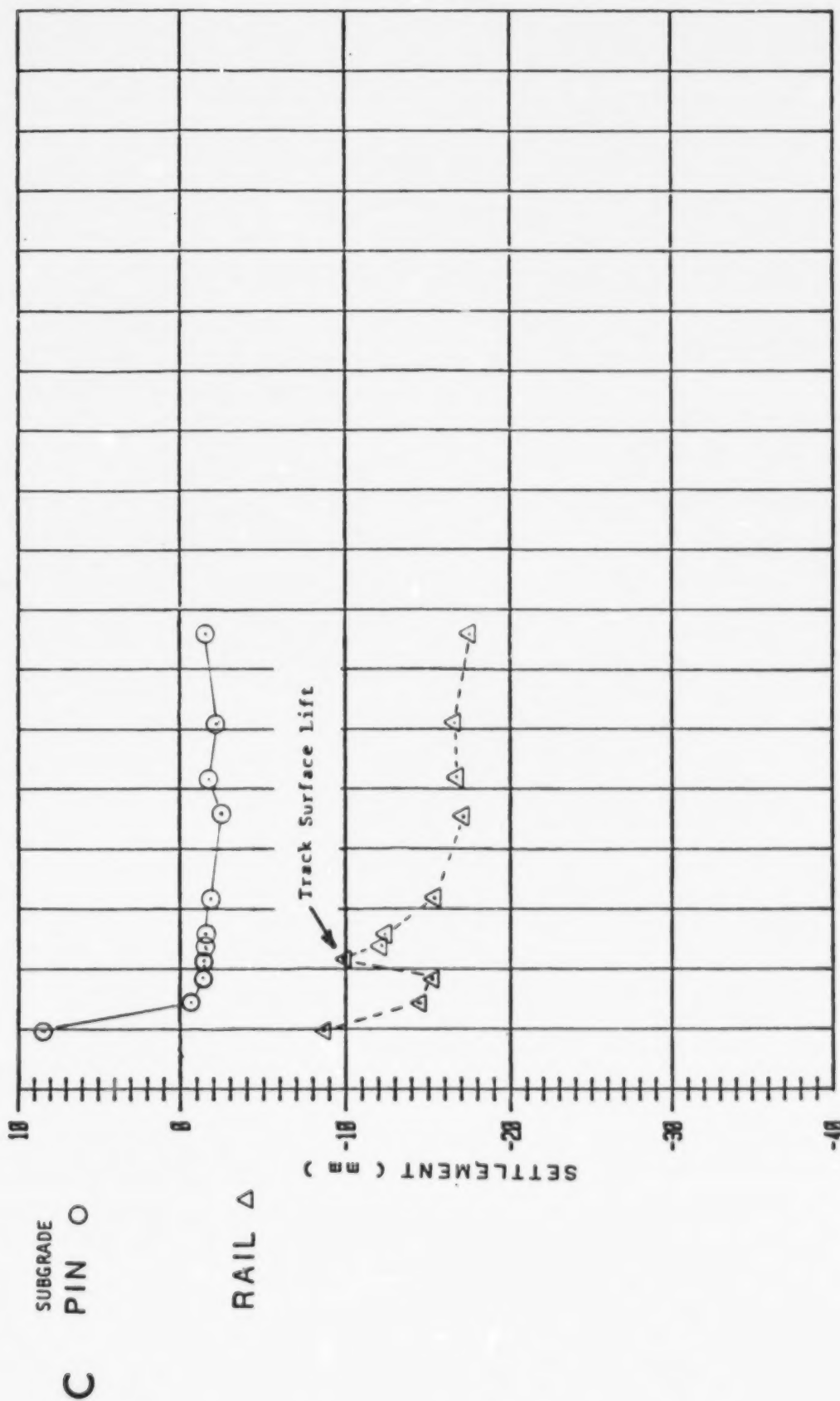


JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC

NOV 1986 - 1987

SETTLEMENT GAUGE READINGS

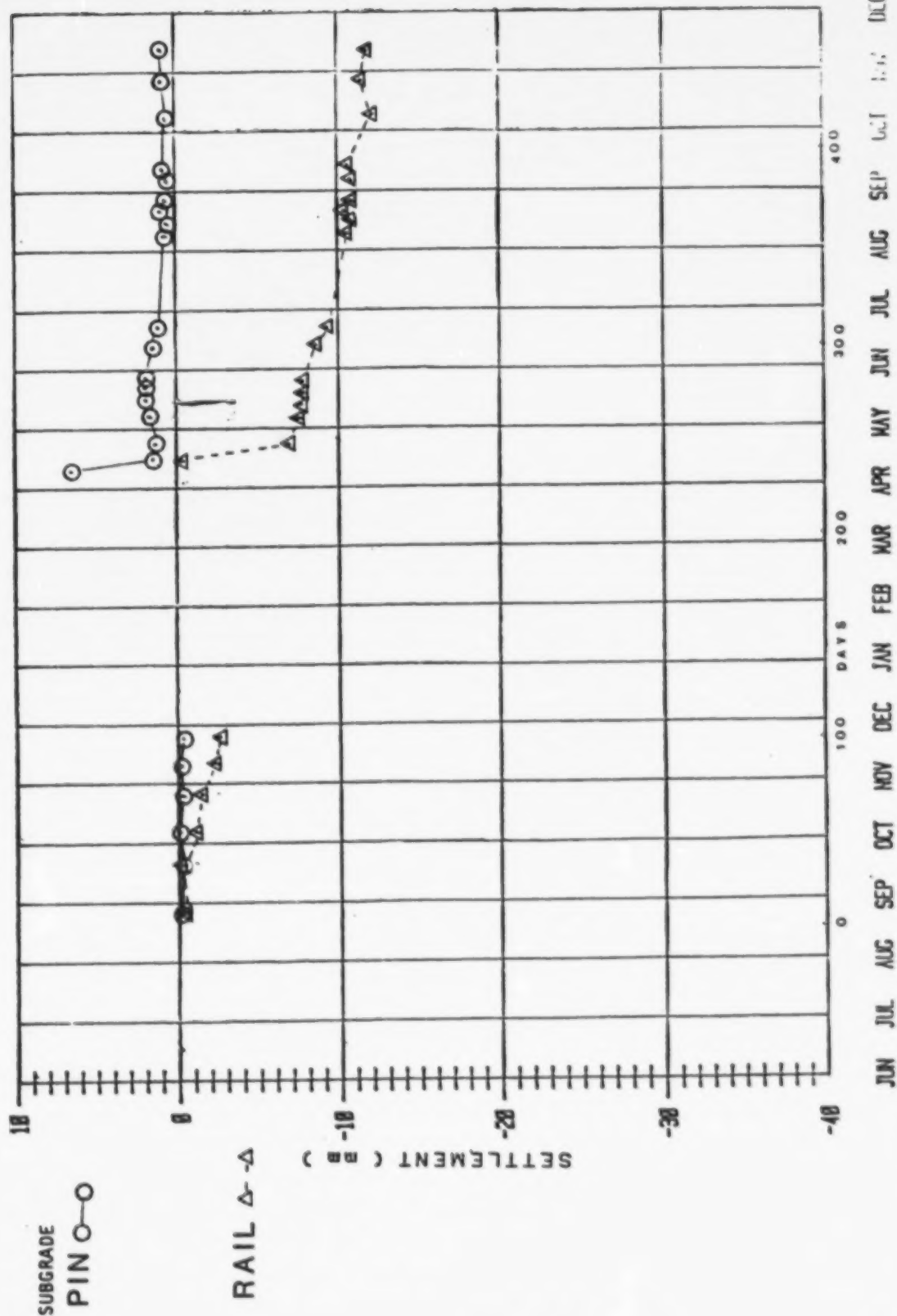
M 118.2 JOLIETTE S/D SITE NO 18



APR MAY JUN JUL AUG SEP OCT

SETTLEMENT GAUGE READINGS

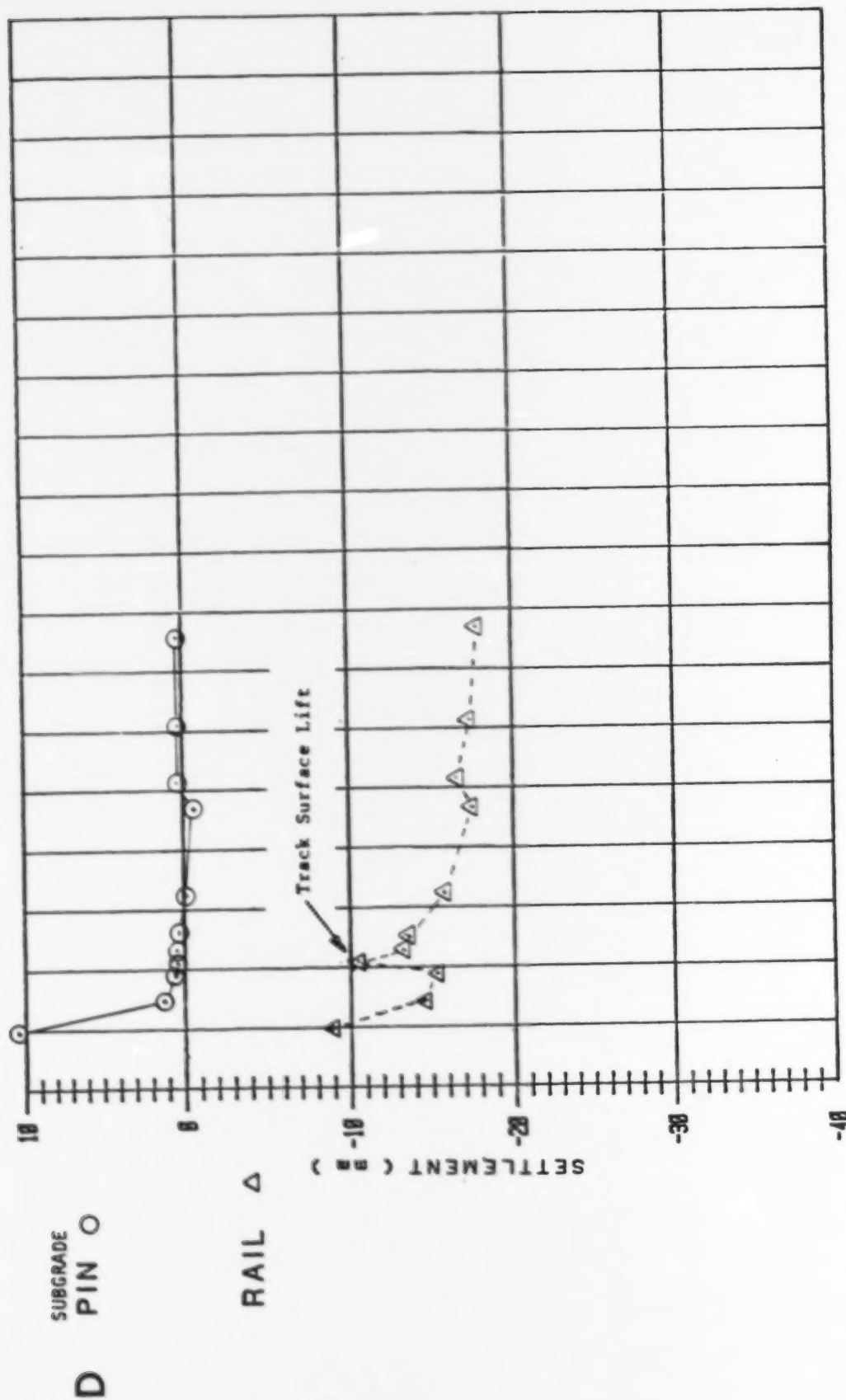
M 118.2 JOLIETTE S/D SITE NO 18-D



MONTHS (1988 - 1991)

SETTLEMENT GAUGE READINGS

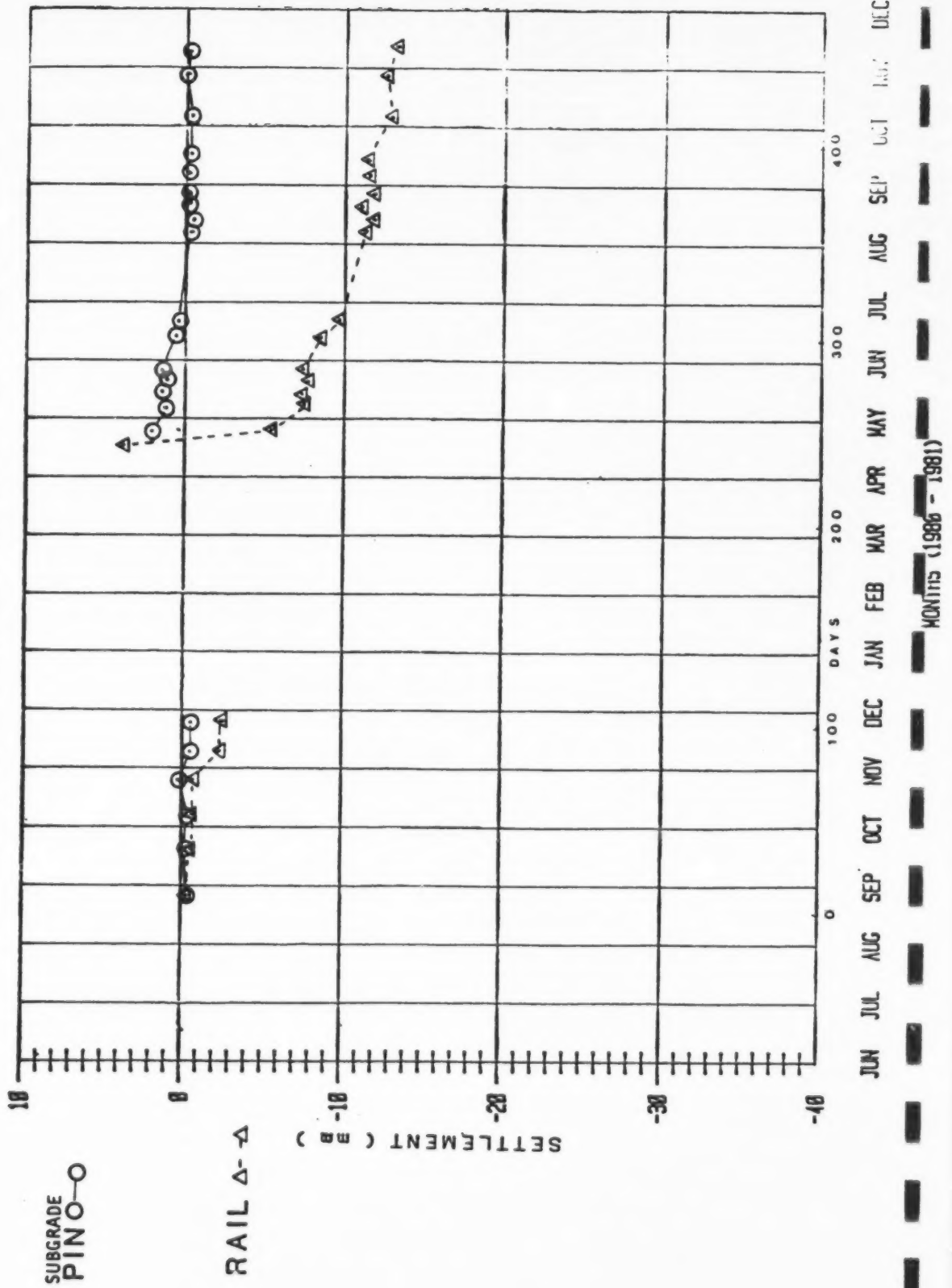
M 118.2 JOLIETTE S/D SITE NO 18



APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

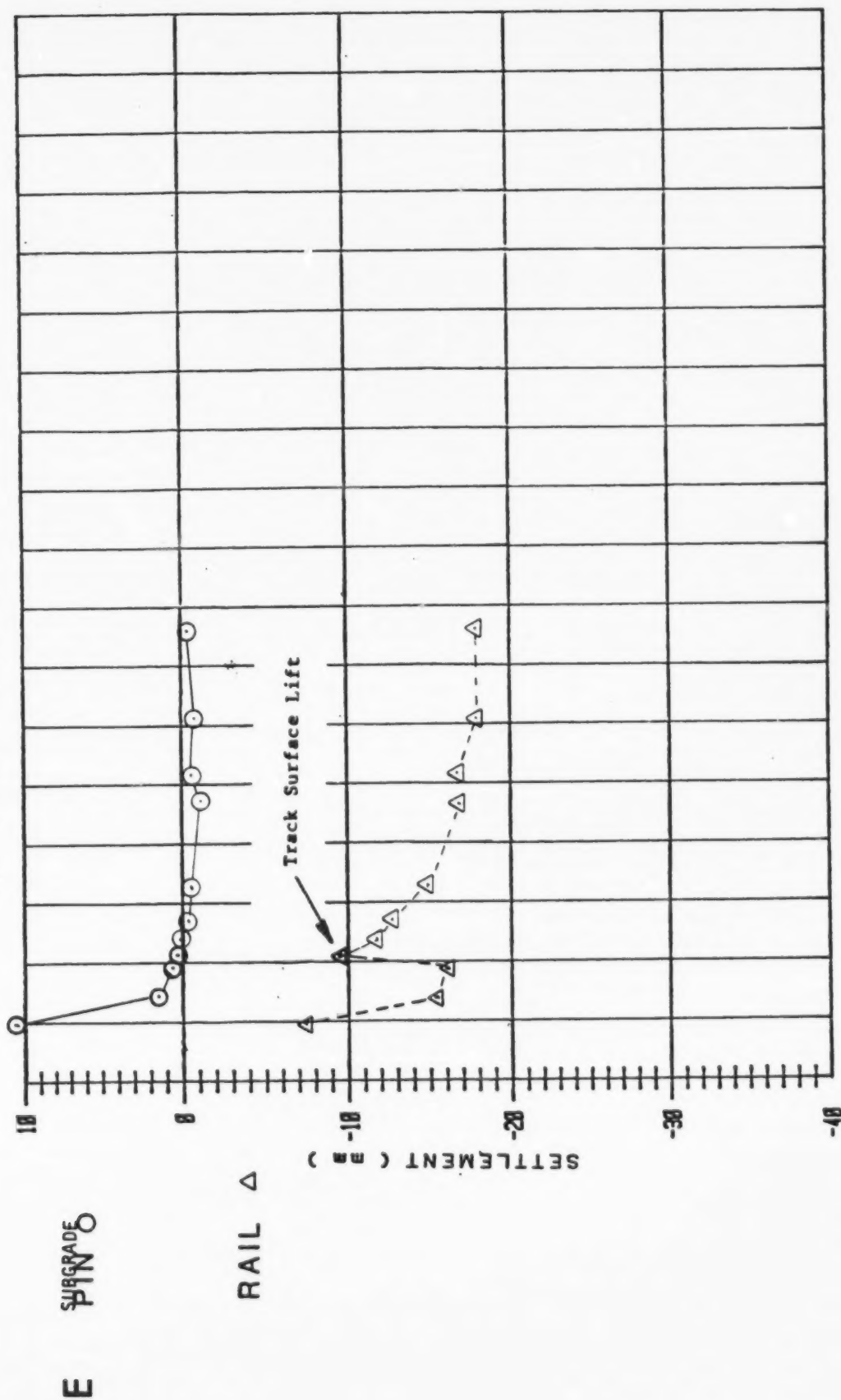
1982 MONTHS

SETTLEMENT GAUGE READINGS M 118.2 JOLIETTE S/D SITE NO 18-E



SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18

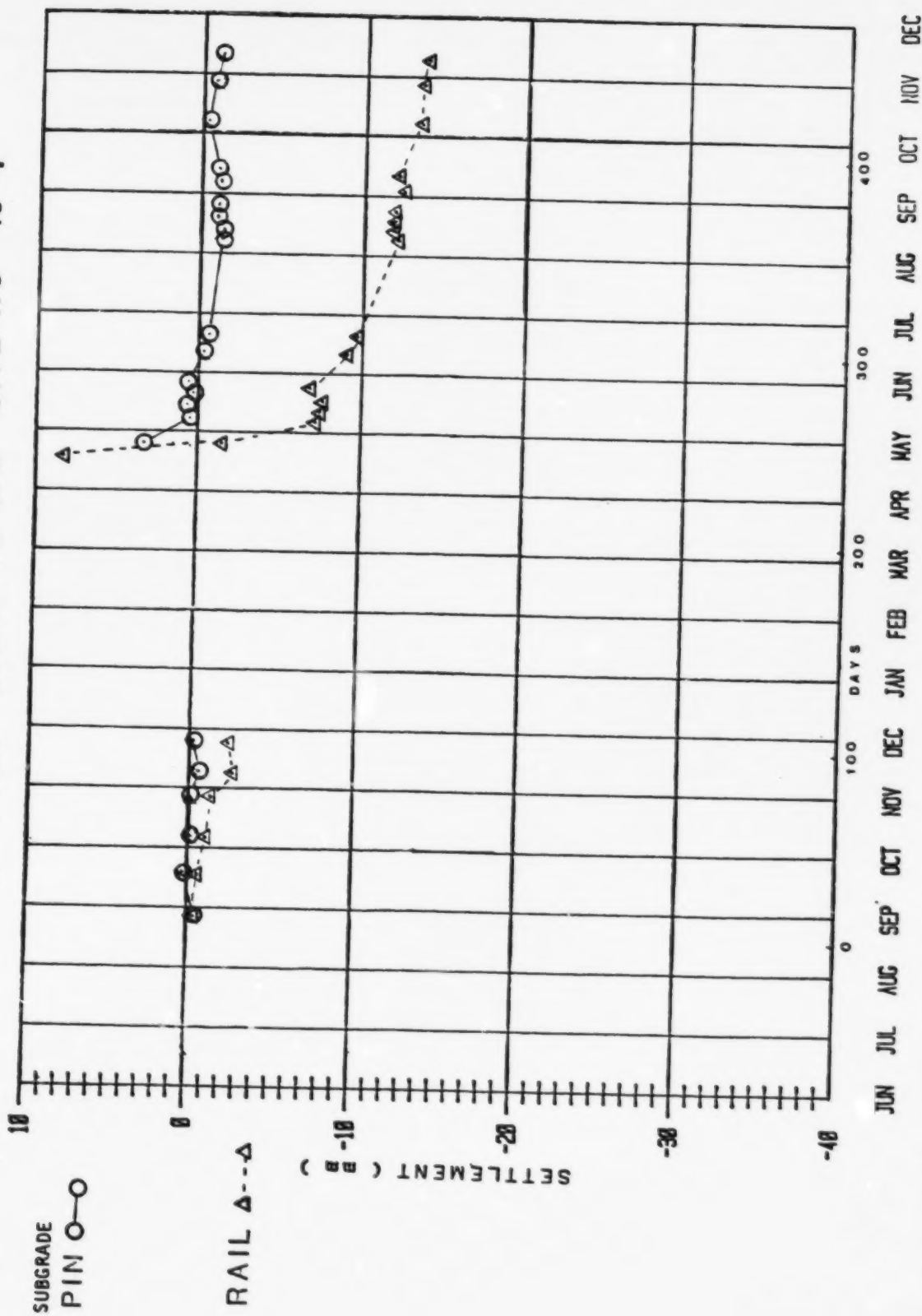


APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

1982 MONTHS

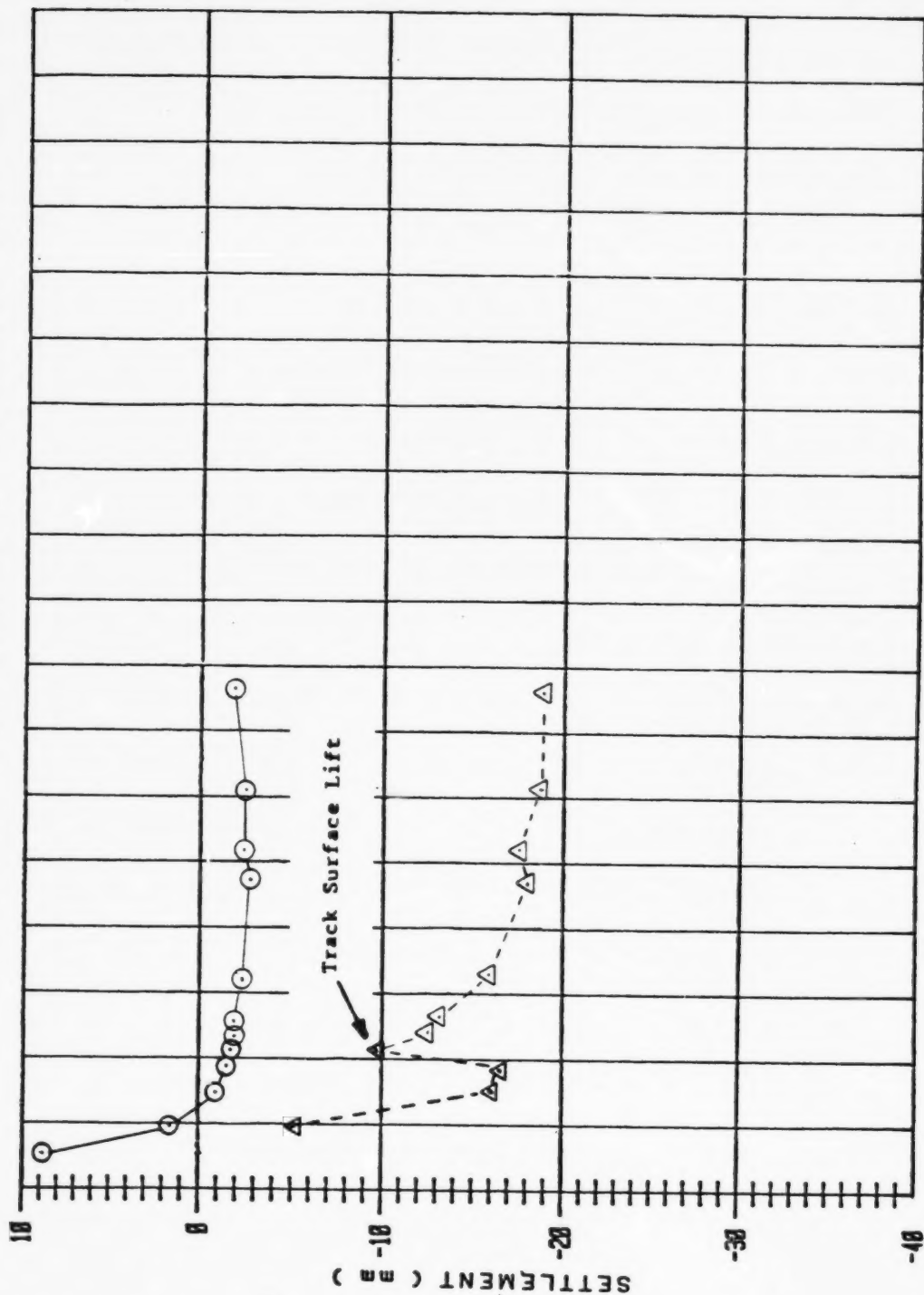
SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18 -F



SETTLEMENT GAUGE READINGS

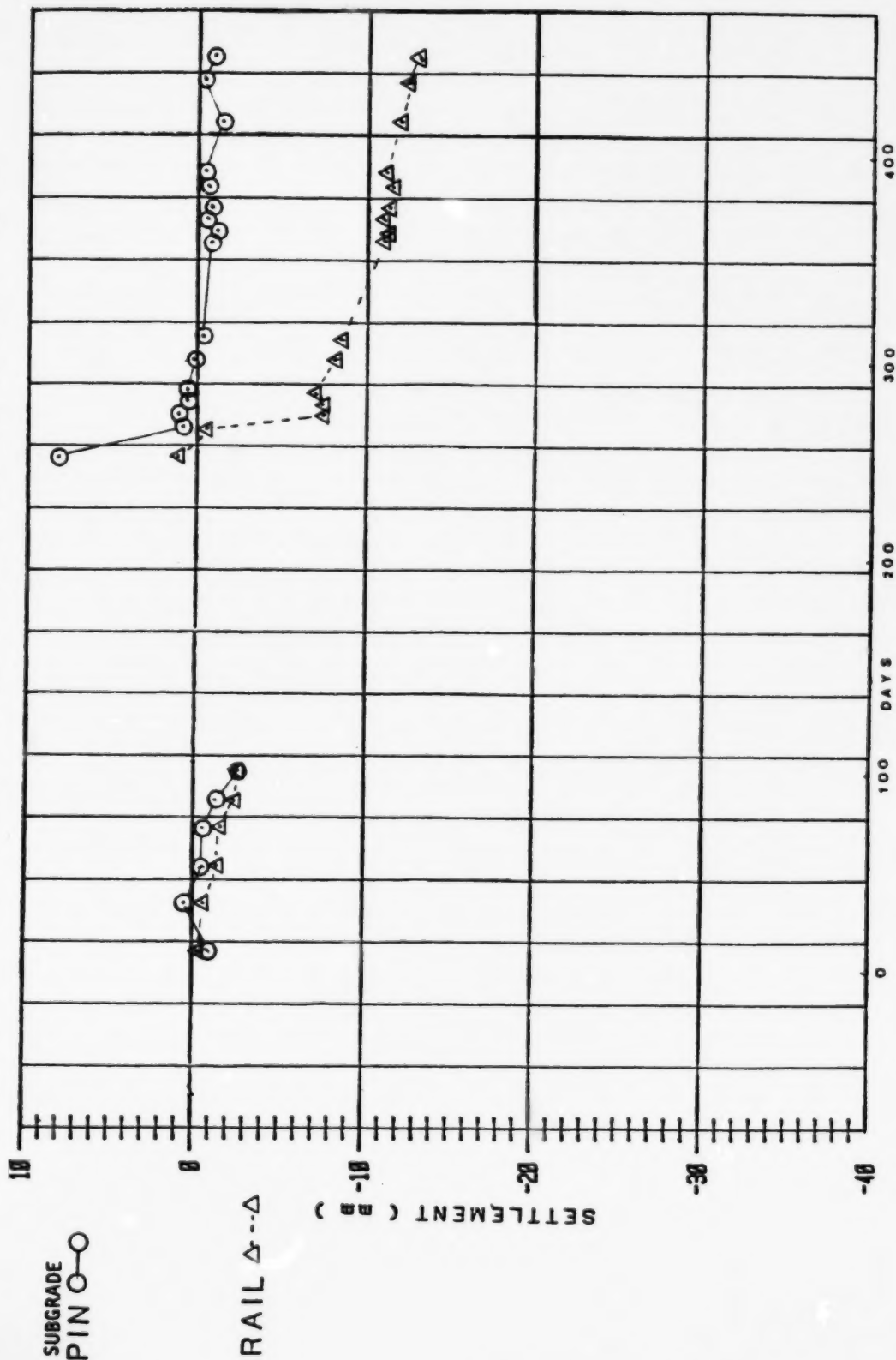
M 118.2 JOLIETTE S/D SITE NO 18



APR MAY JUN JUL AUG SEP OCT

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-G

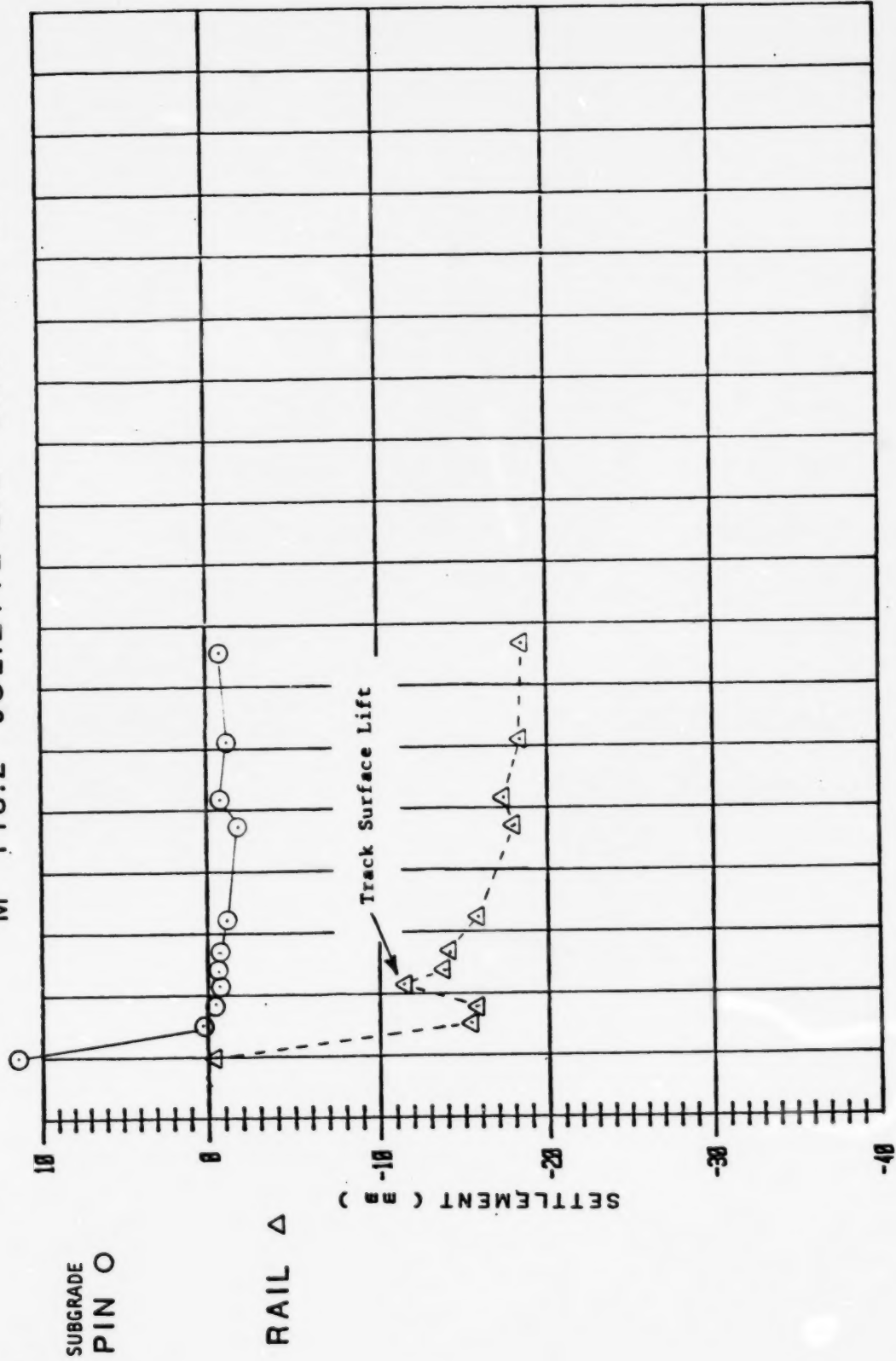


JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC

1961 (152)

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18

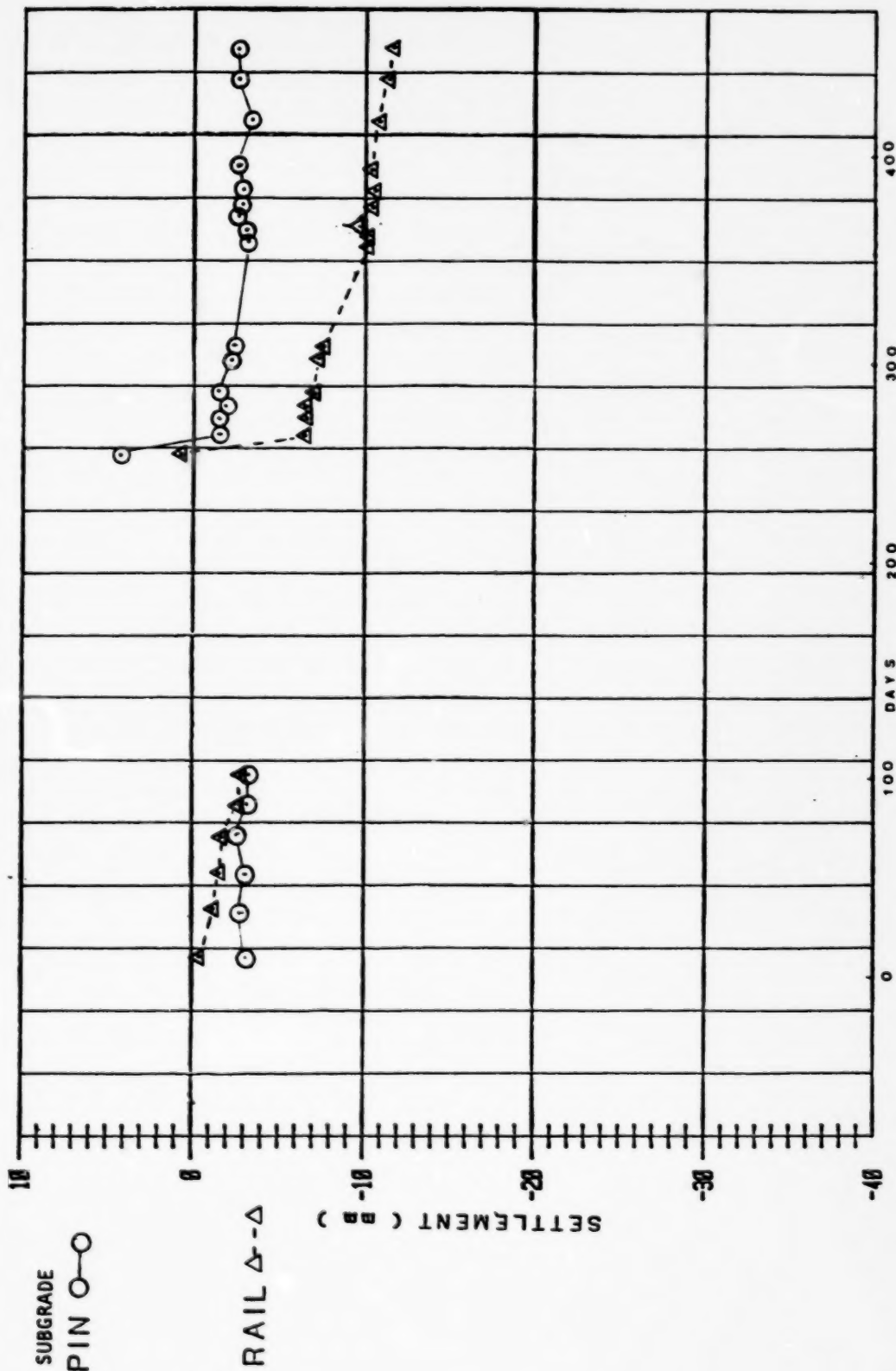


APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

1982 MONTHS

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-H



JUN JUL AUG SEP OCT NOV DEC

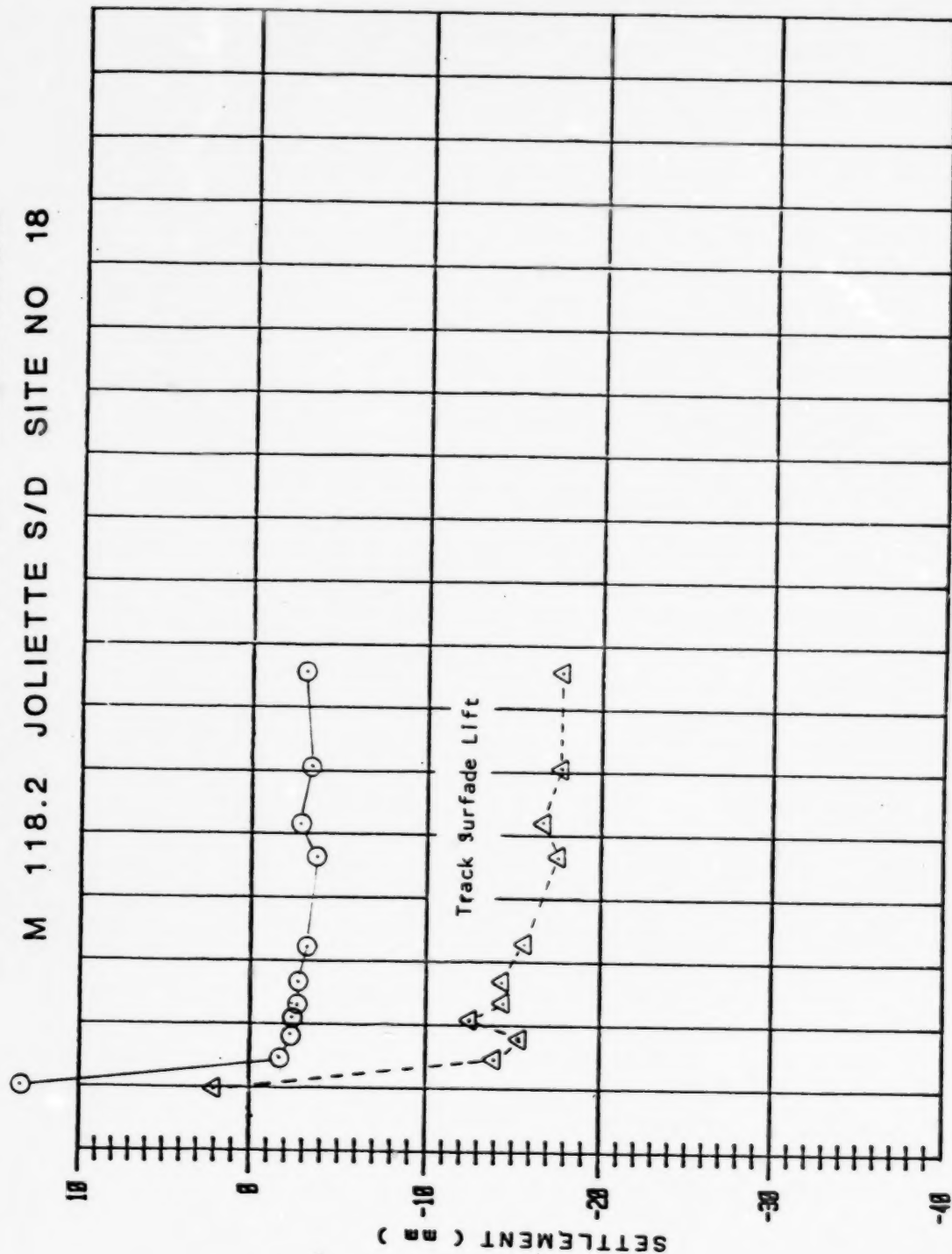
MONTHS (1900-1901)

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18

H
SUBGRADE
PINO

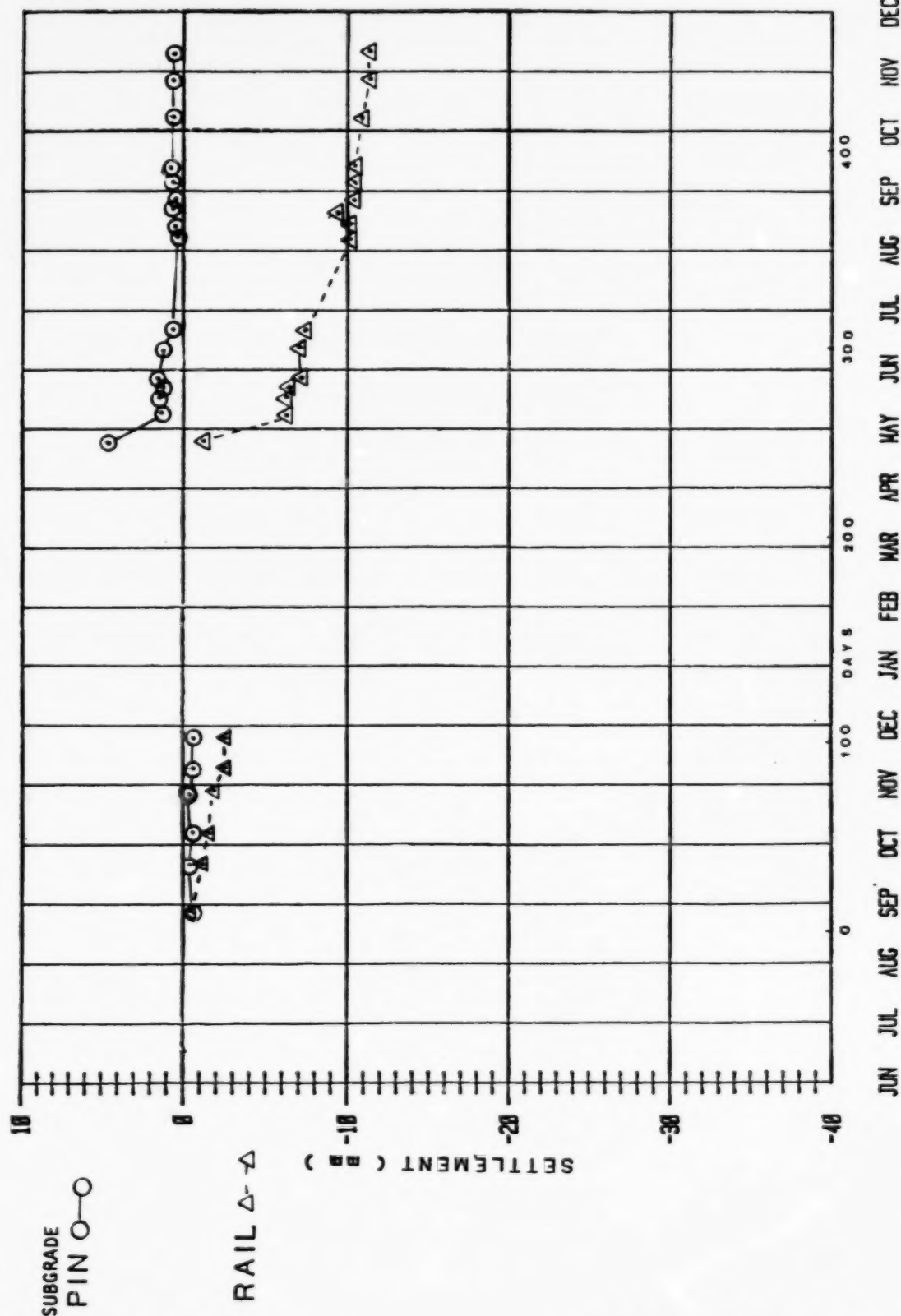
RAIL Δ



APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

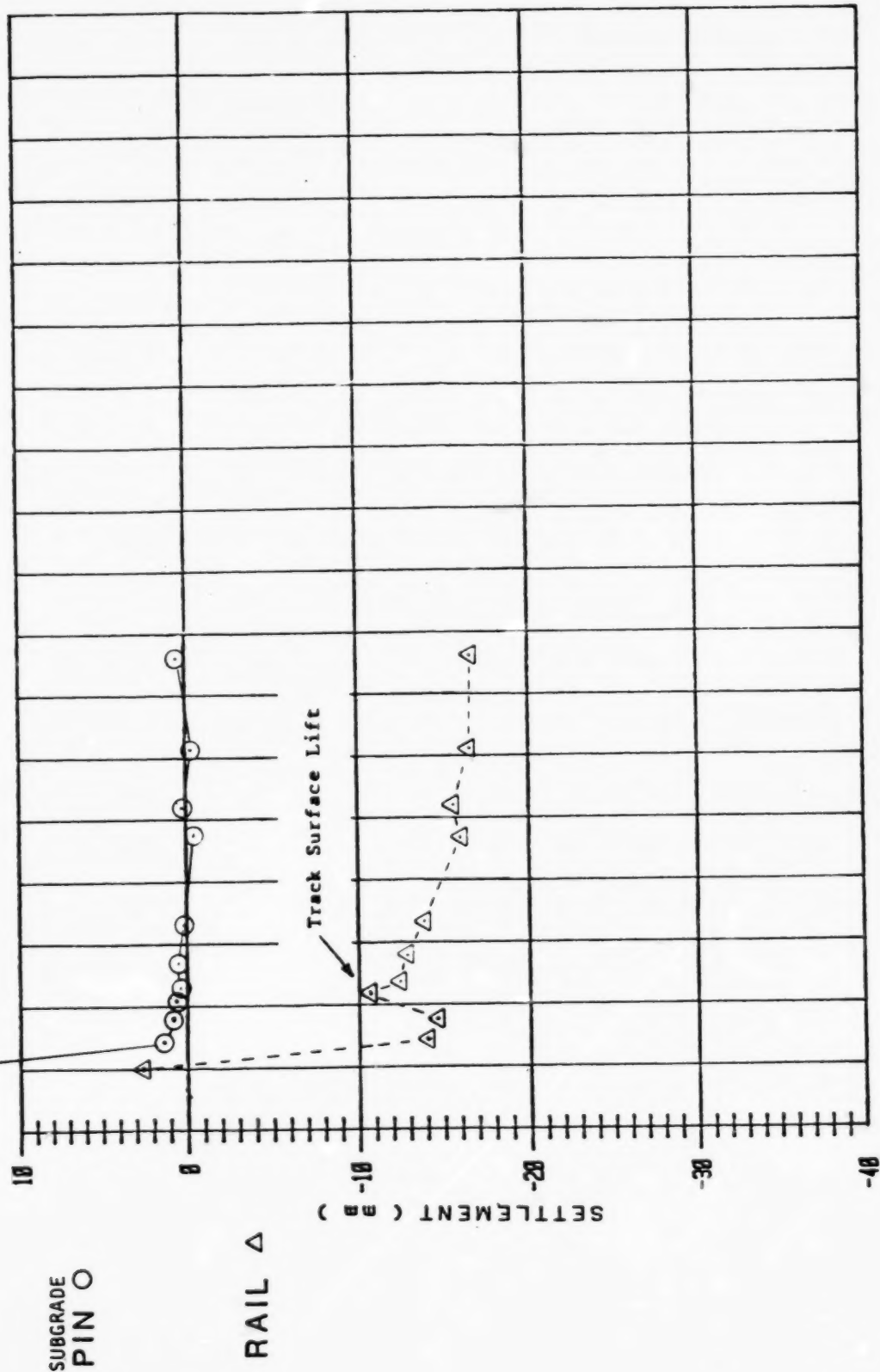
SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-i



SETTLEMENT GAUGE READINGS

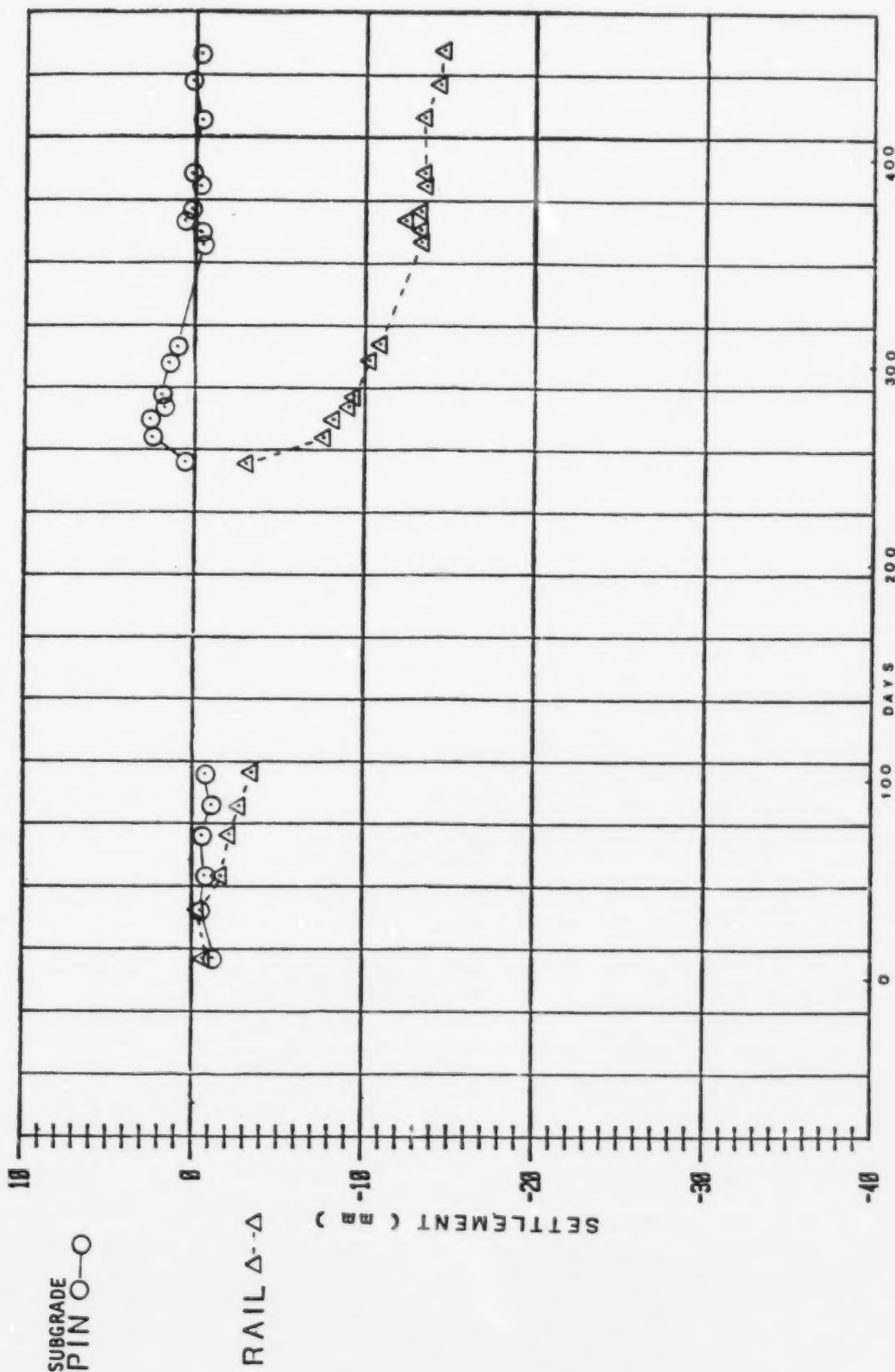
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APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-J

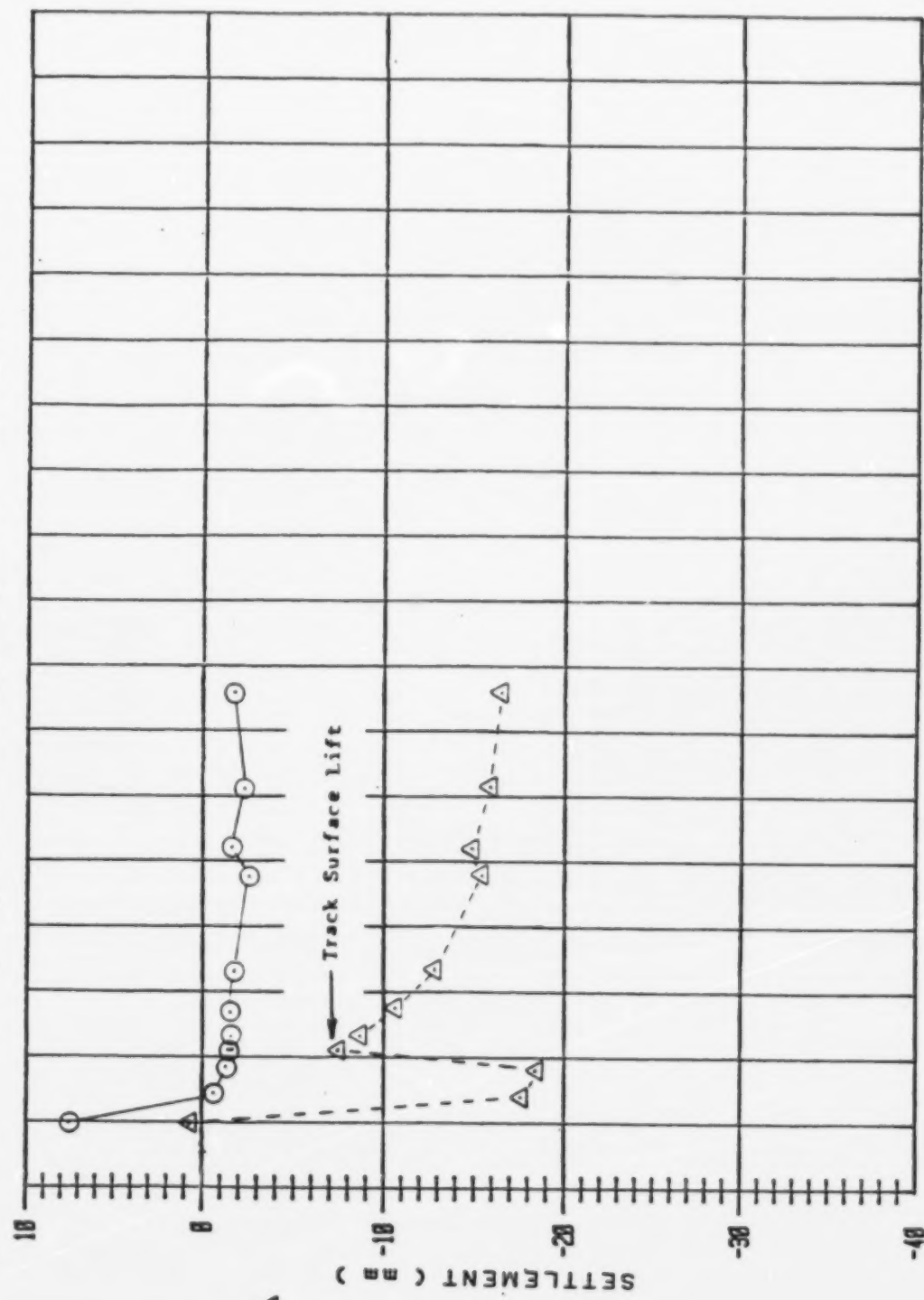


JUN JUL AUG SEP OCT NOV DEC

MONTHS (1962-1963)

SETTLEMENT GAUGE READINGS

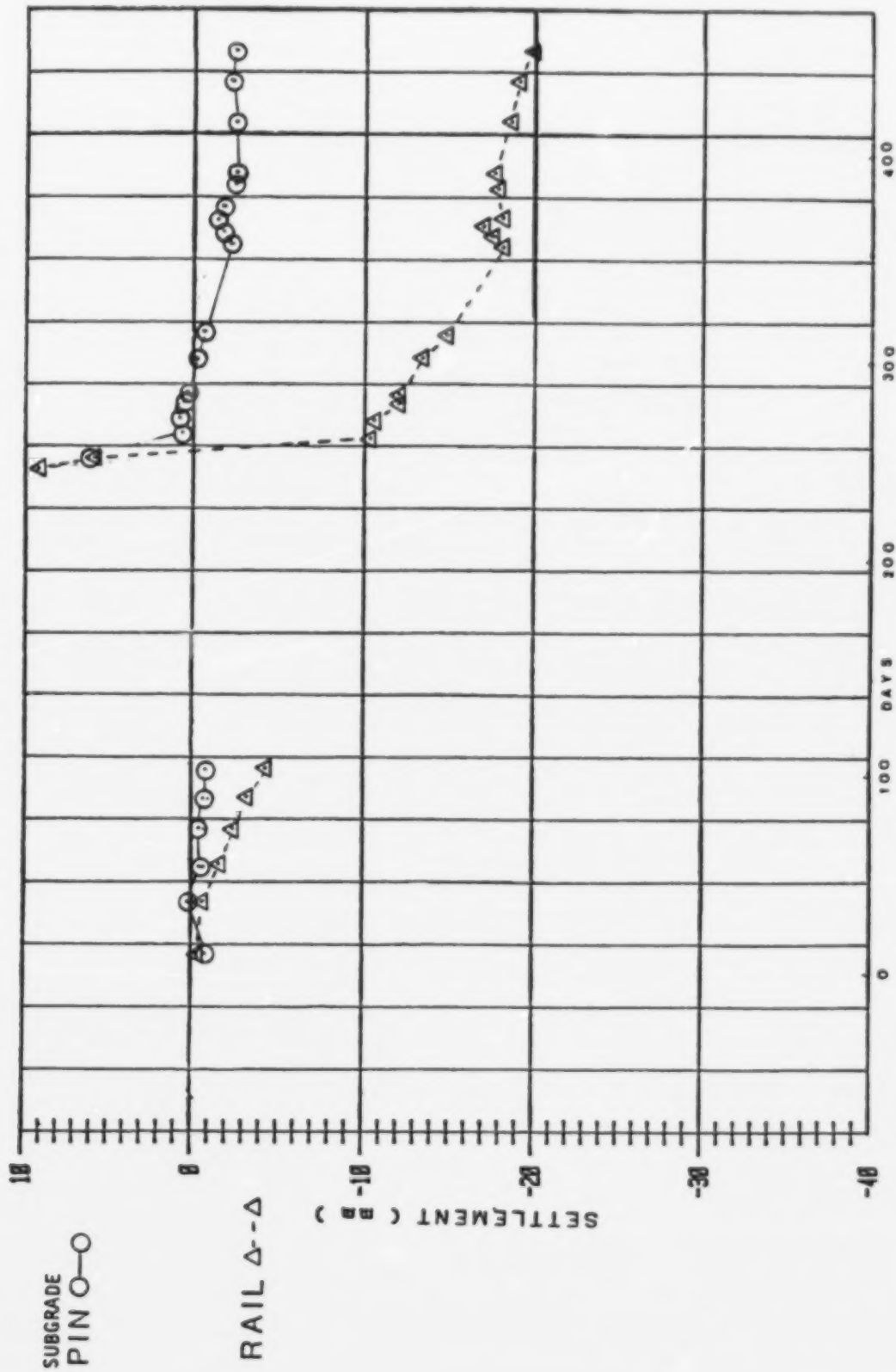
M 118.2 JOLIETTE S/D SITE NO 18



APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-K

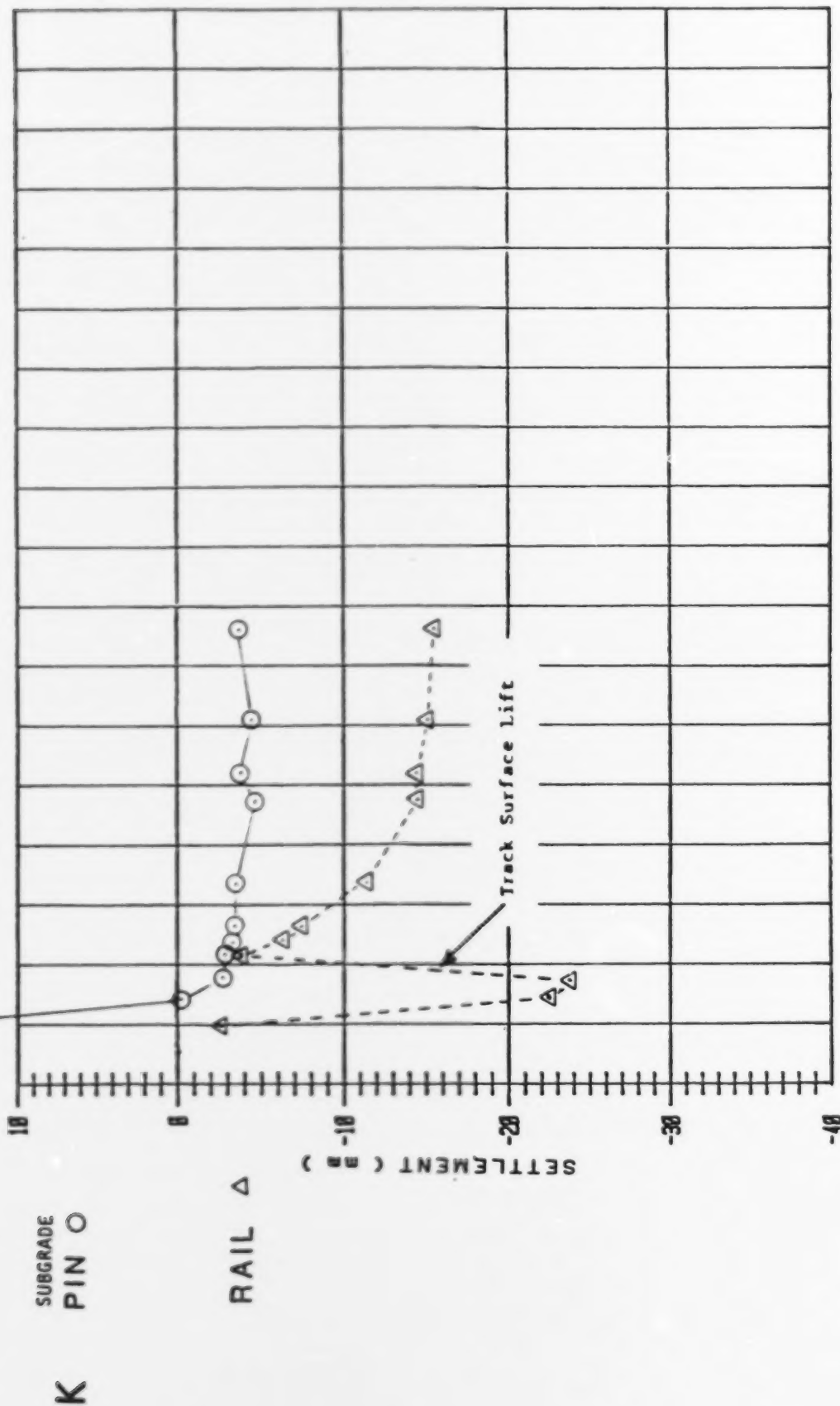


JUN JUL AUG SEP OCT NOV DEC

MONTHS (1900-1901)

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18

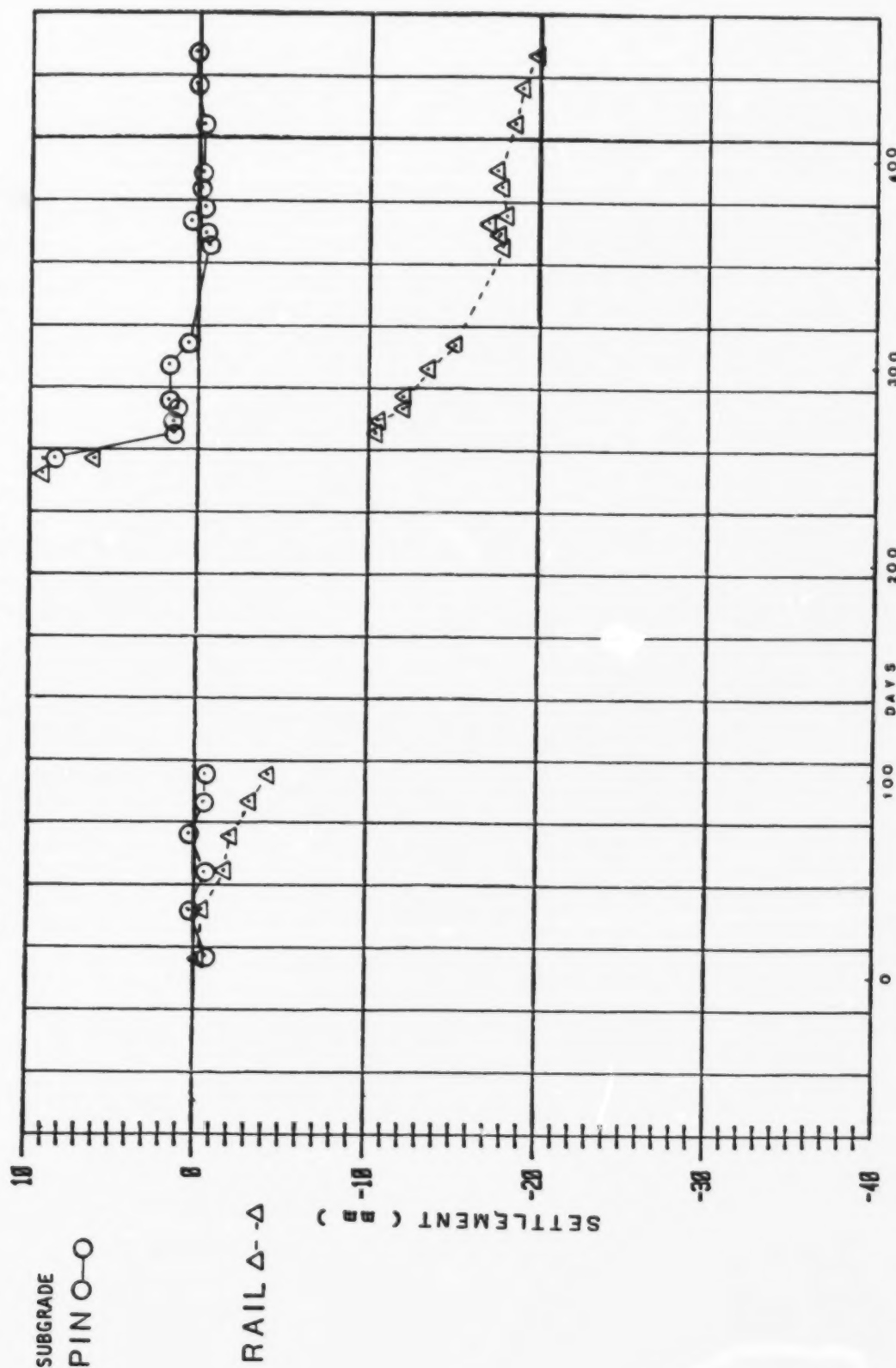


APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

1982 MONTHS

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-L

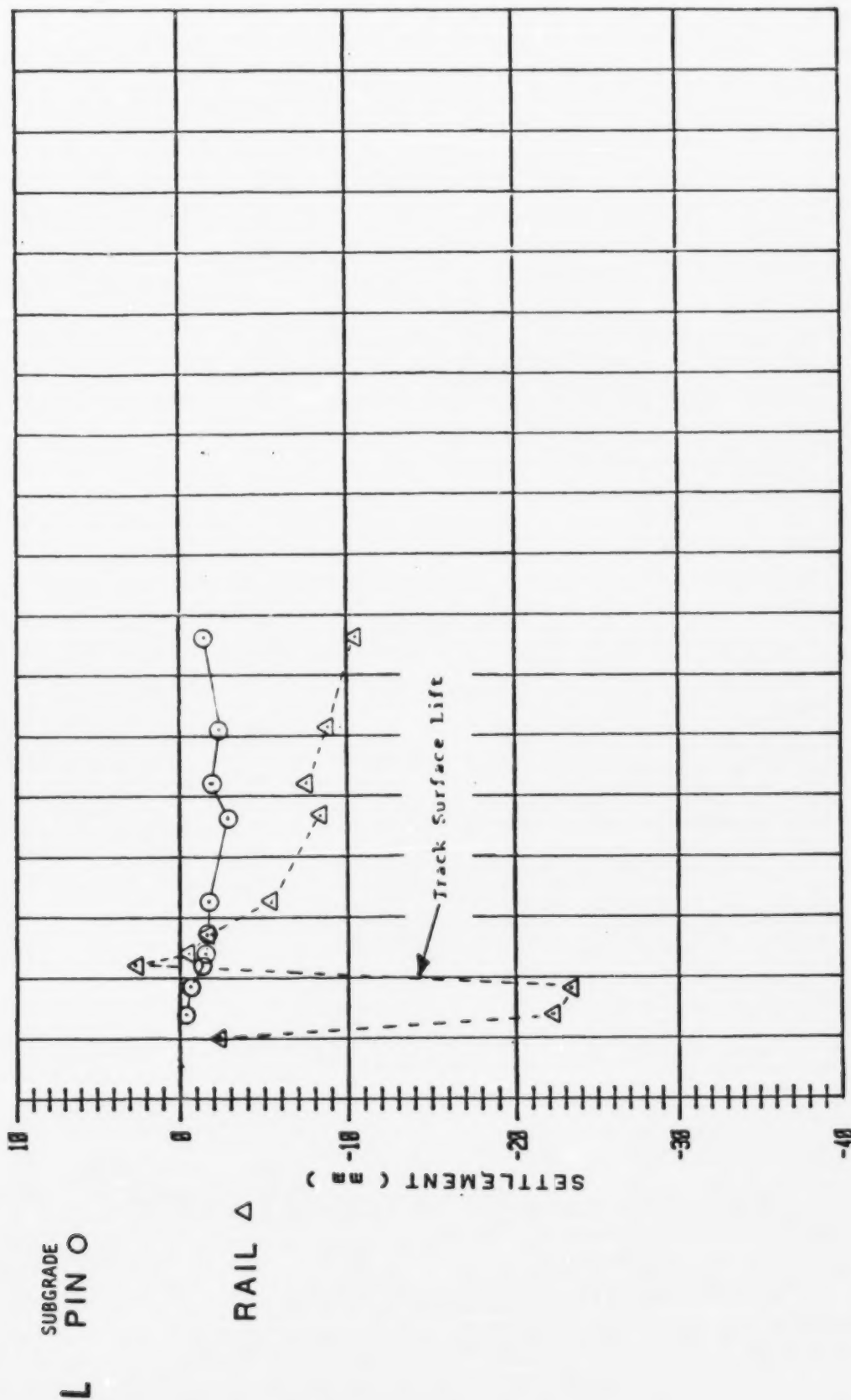


JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC

1900 1901

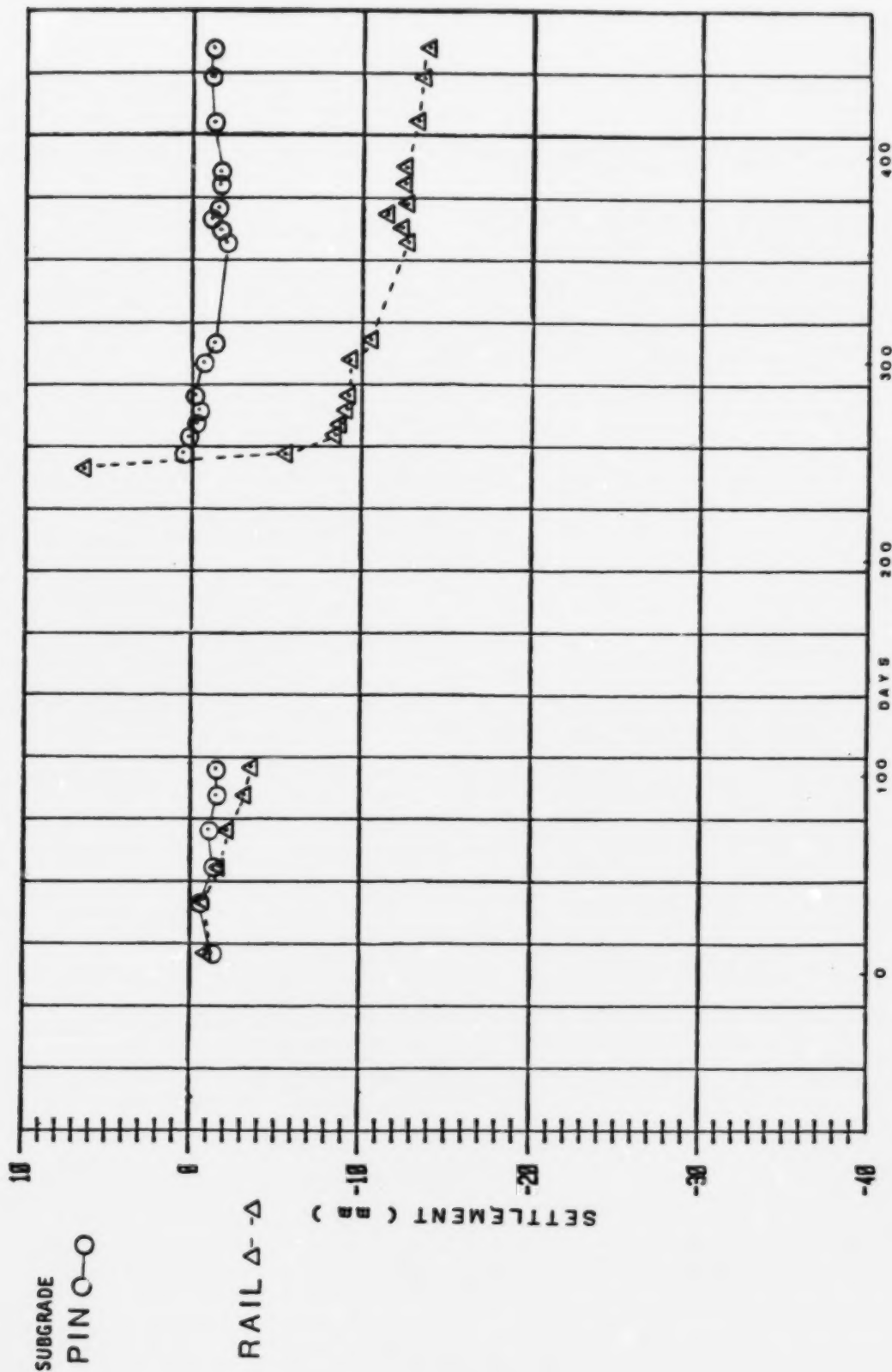
SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18



SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18 - M

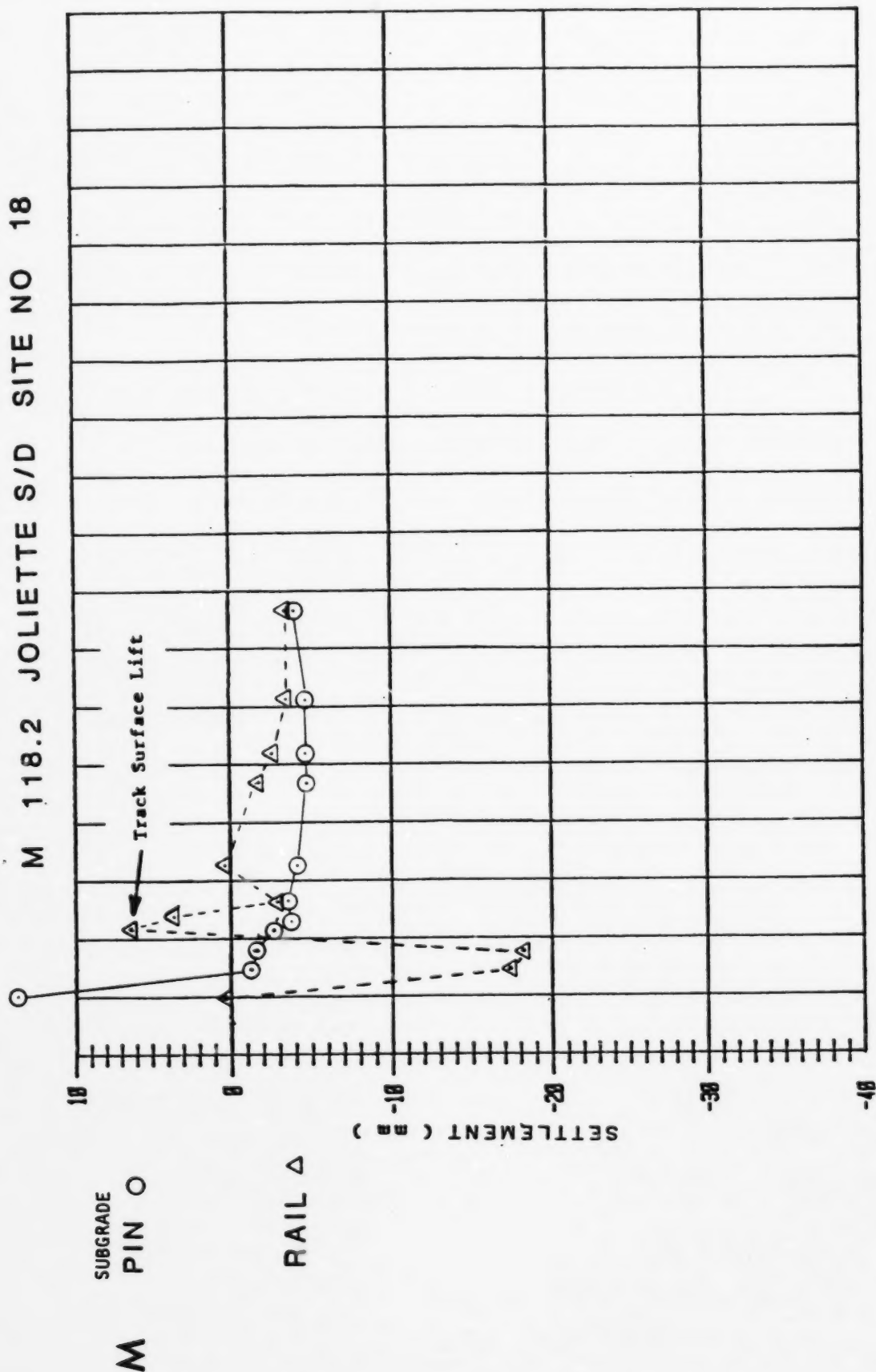


JUN JUL AUG SEP OCT NOV DEC

MONTHS (1980 - 1981)

SETTLEMENT GAUGE READINGS

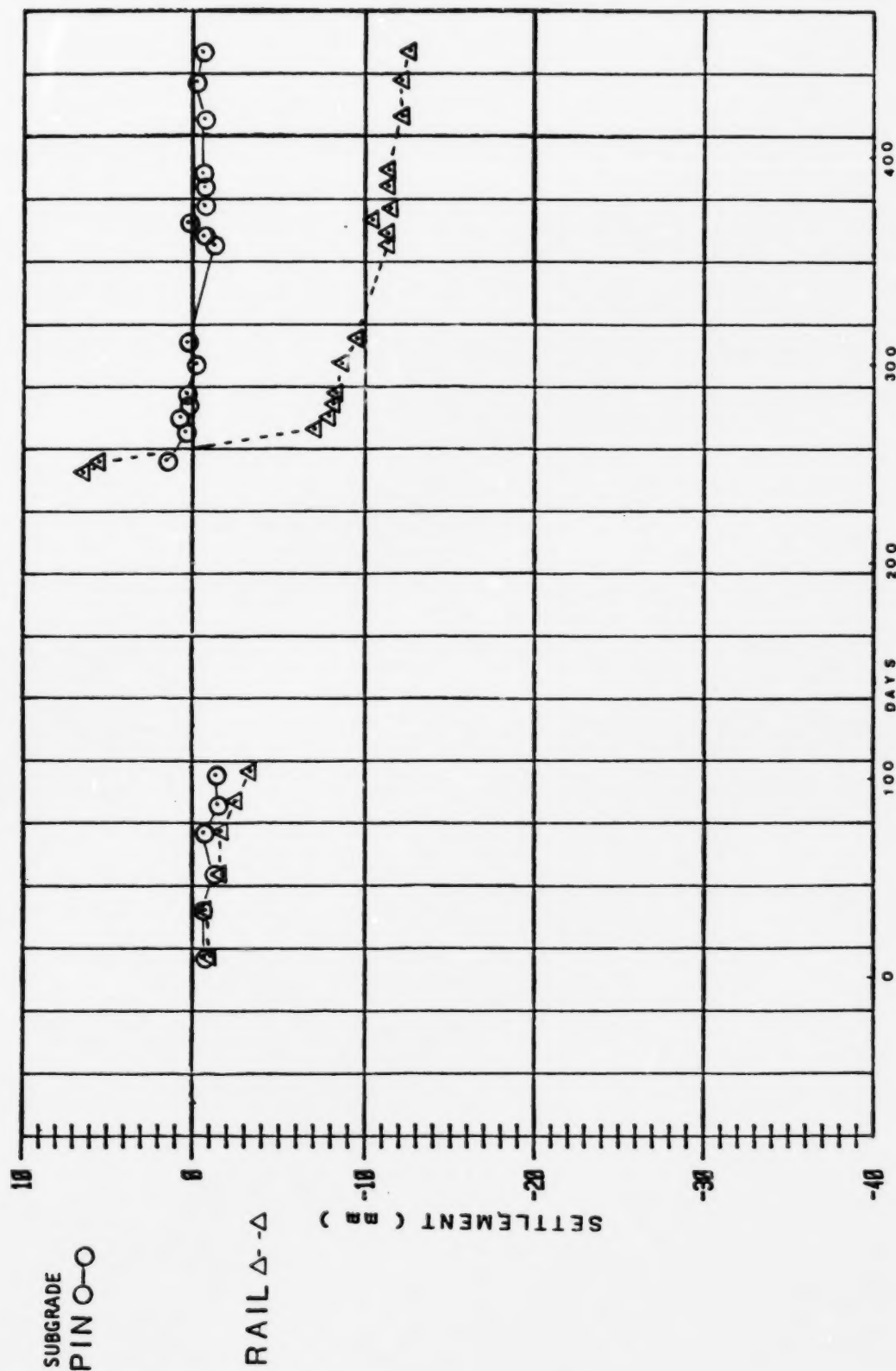
M 118.2 JOLIETTE S/D SITE NO 18



APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18 - M

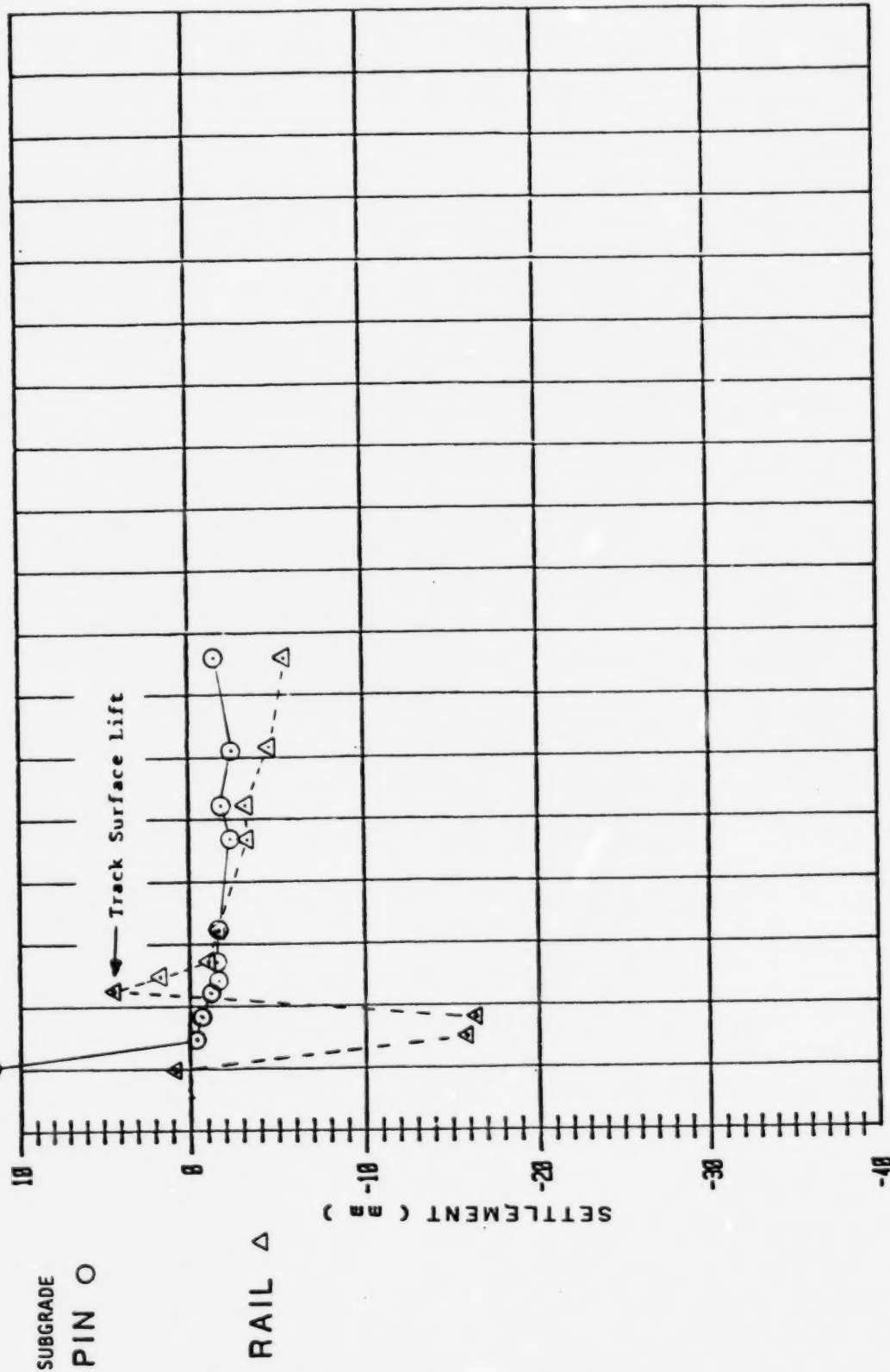


JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC

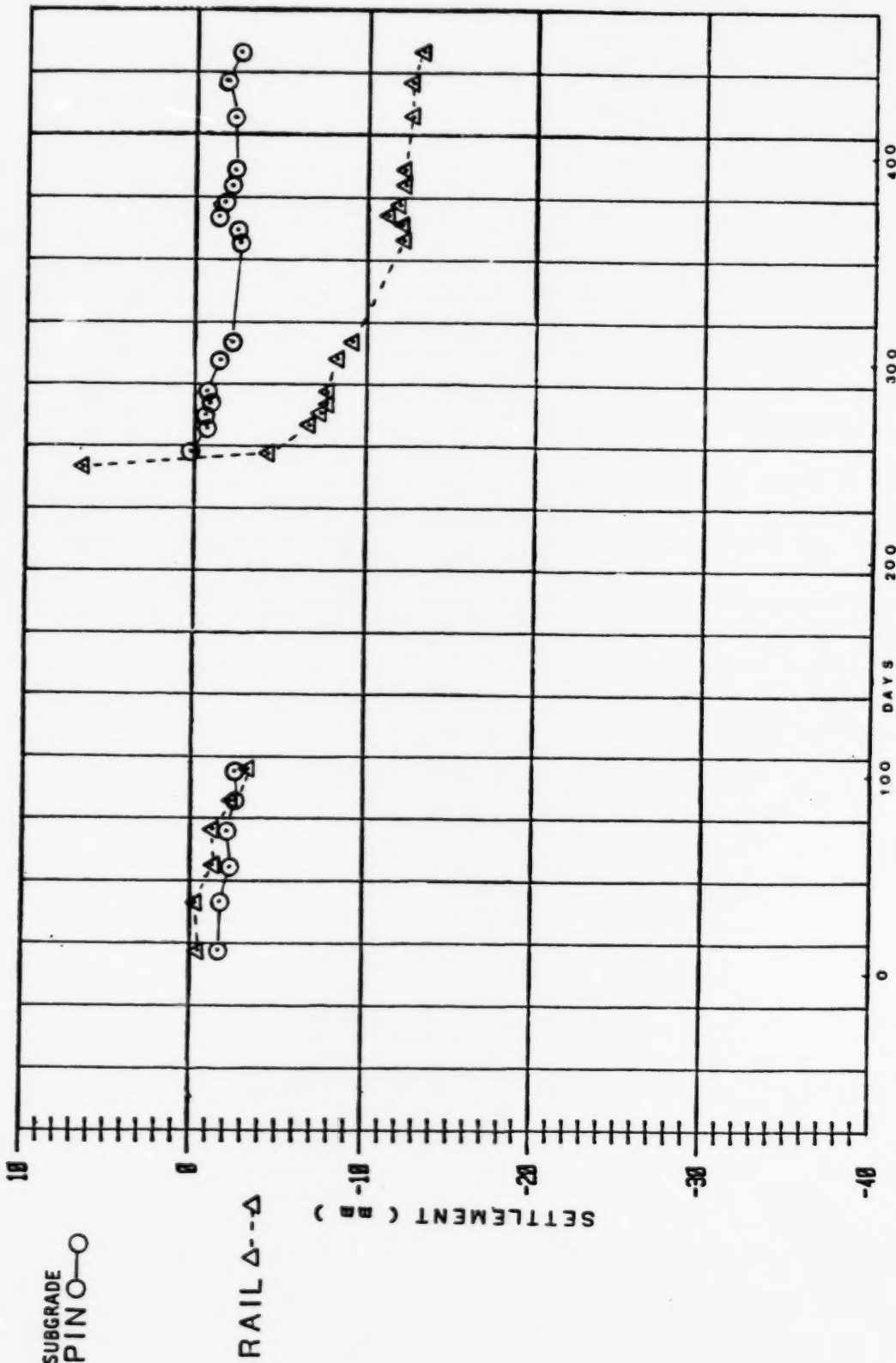
1981

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18



SETTLEMENT GAUGE READINGS M 118.2 JOLIETTE S/D SITE NO 18-0

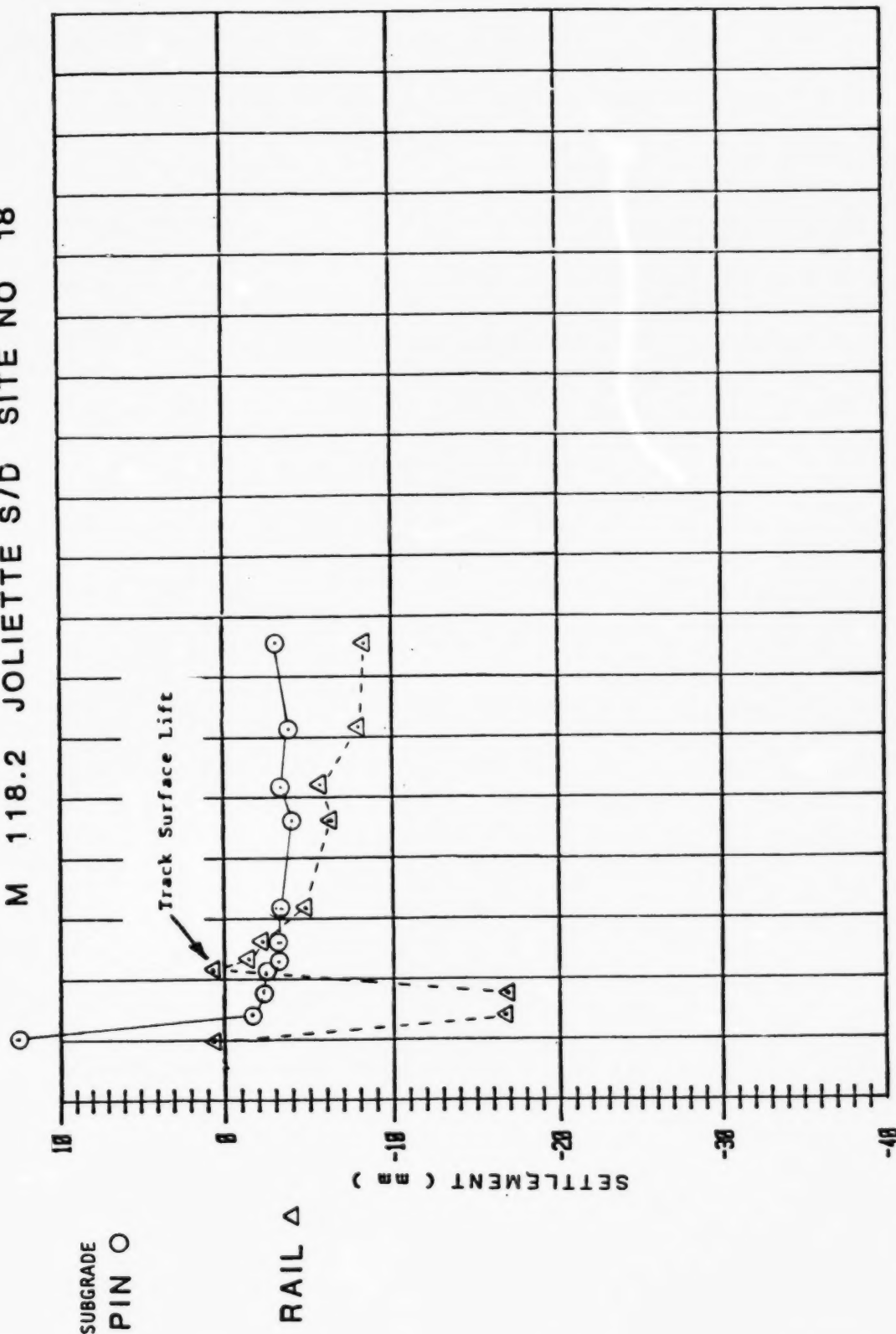


JUN JUL AUG SEP OCT NOV DEC

1911

SETTLEMENT GAUGE READINGS

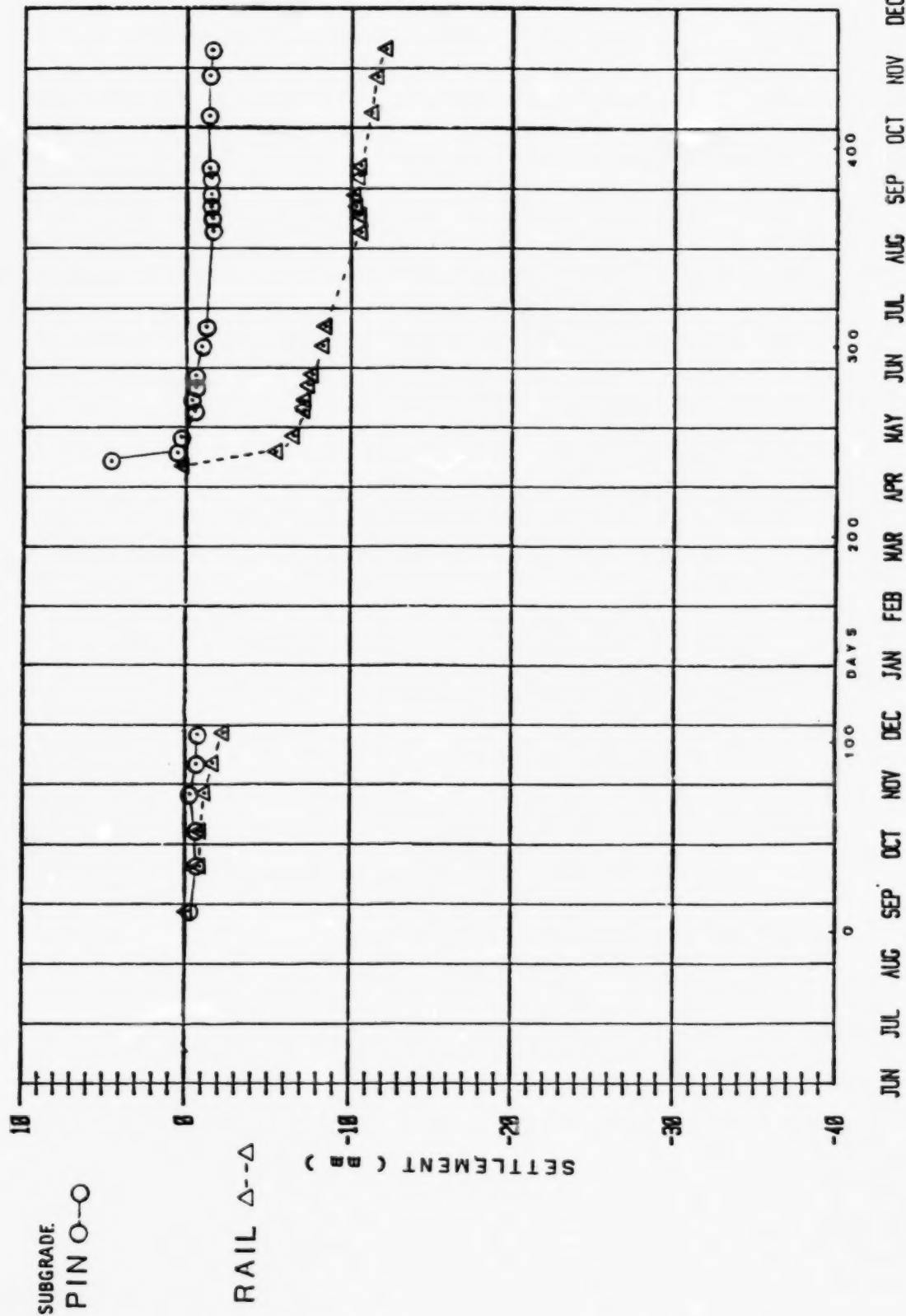
M 118.2 JOLIETTE S/D SITE NO 18



APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

SETTLEMENT GAUGE READINGS

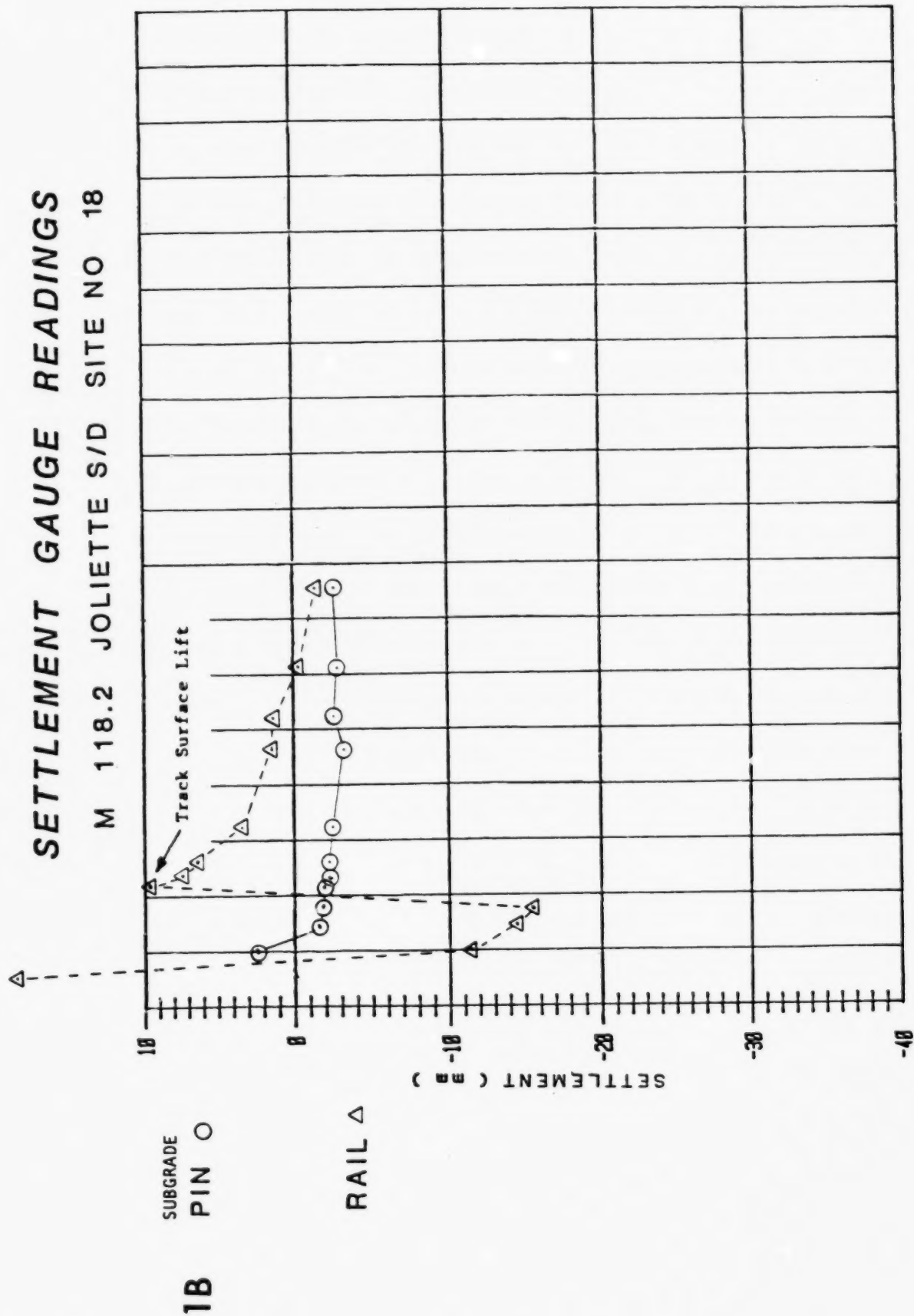
M 118.2 JOLIETTE S/D SITE NO 18-1B



MAY 1961

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18



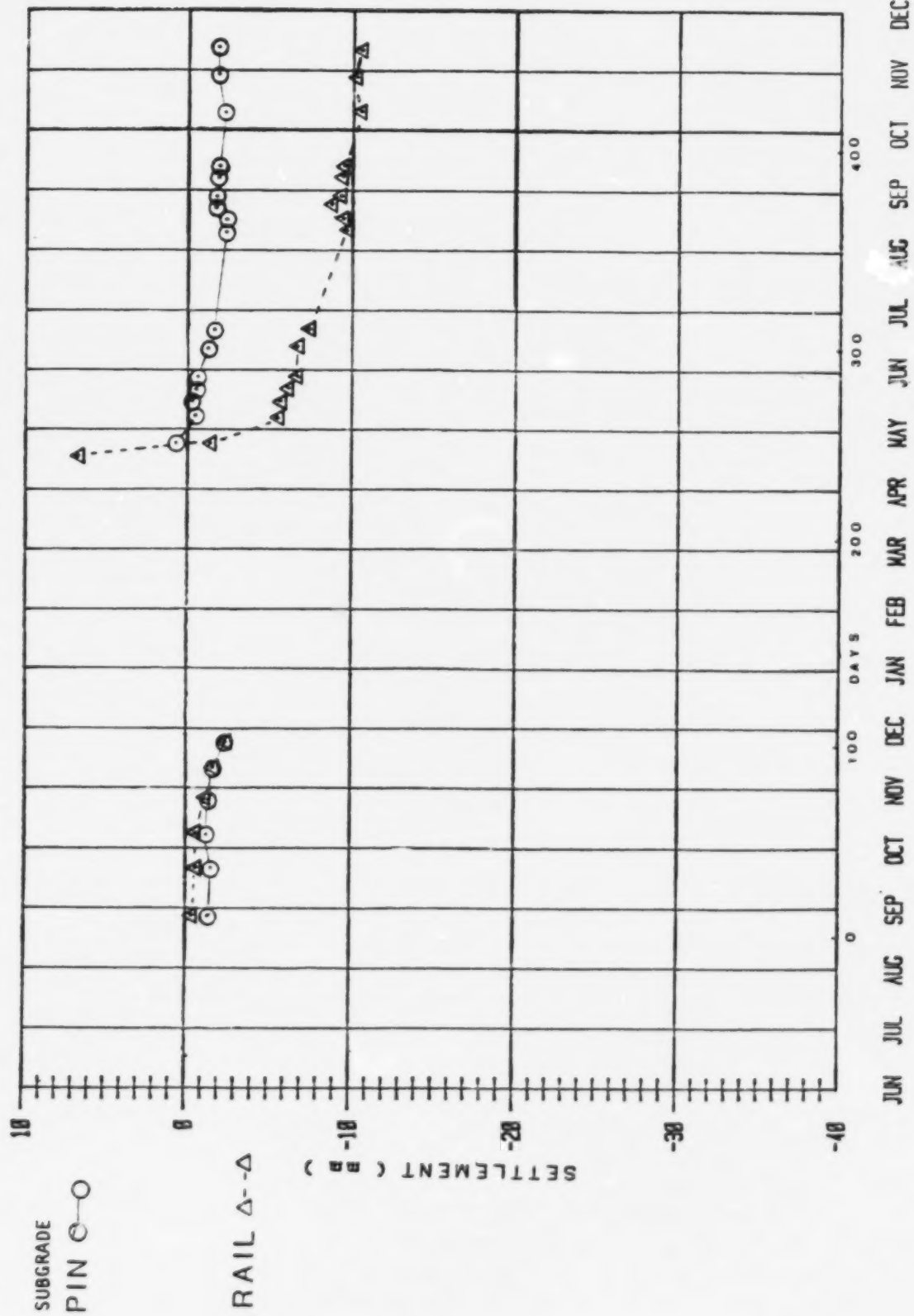
SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

113

1982 MONTHS

SETTLEMENT GAUGE READINGS

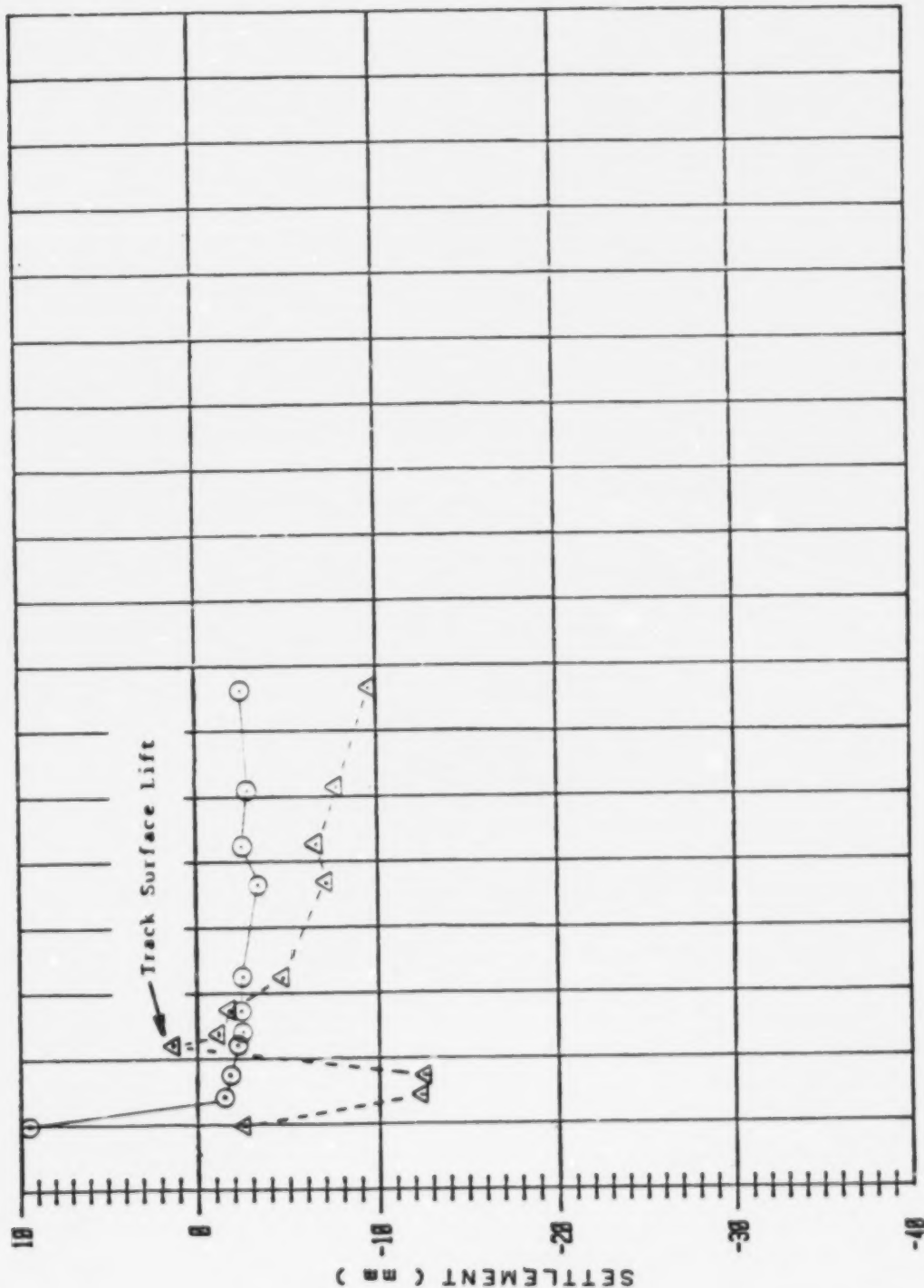
M 118.2 JOLIETTE S/D SITE NO 18-1E



1961

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18



1 E
SUBGRADE
PINO

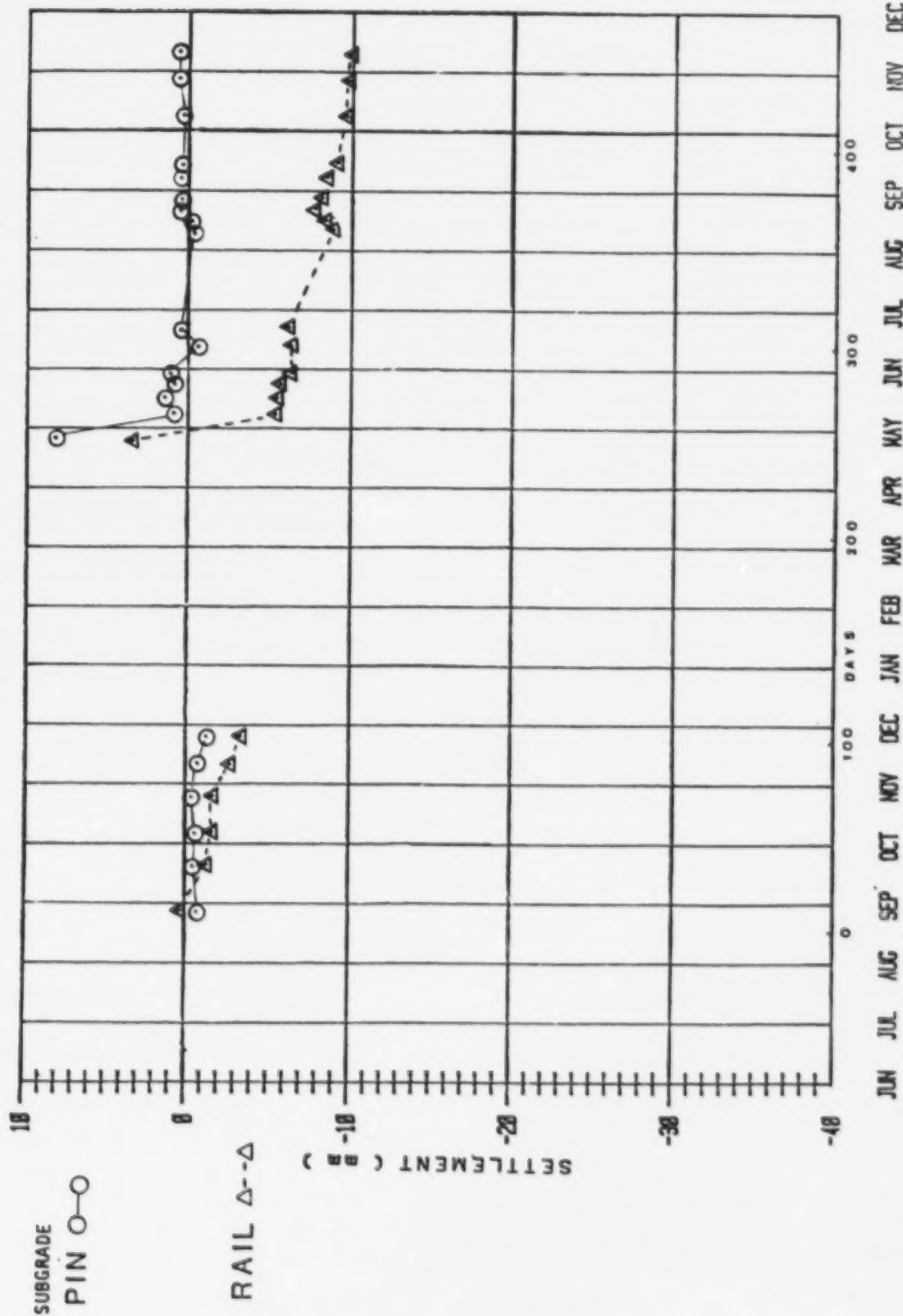
RAIL Δ

APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

1982 MONTHS

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-1H



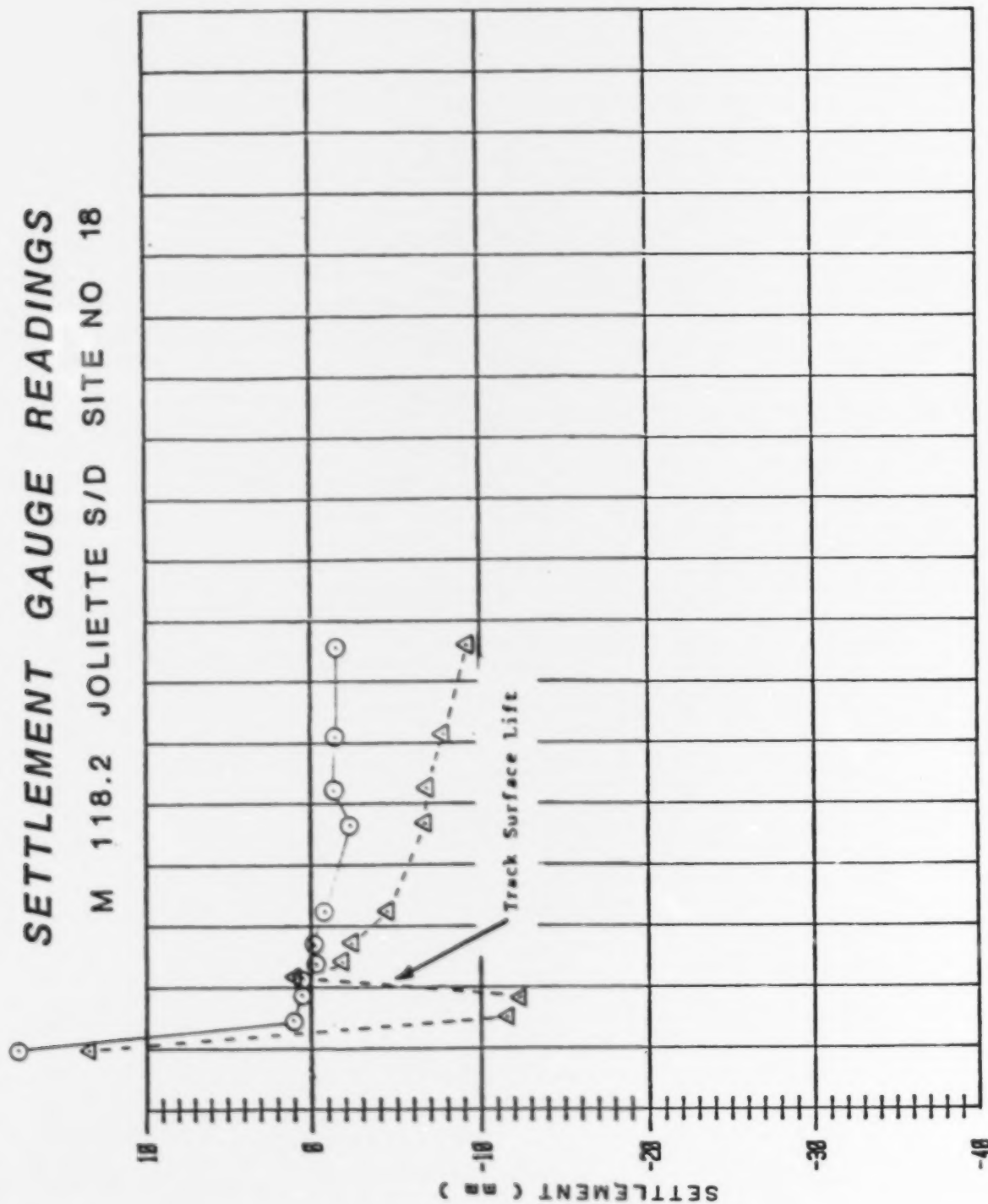
MOBILE (18-1) 1981

SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18

1H
SUBGRADE
PINO

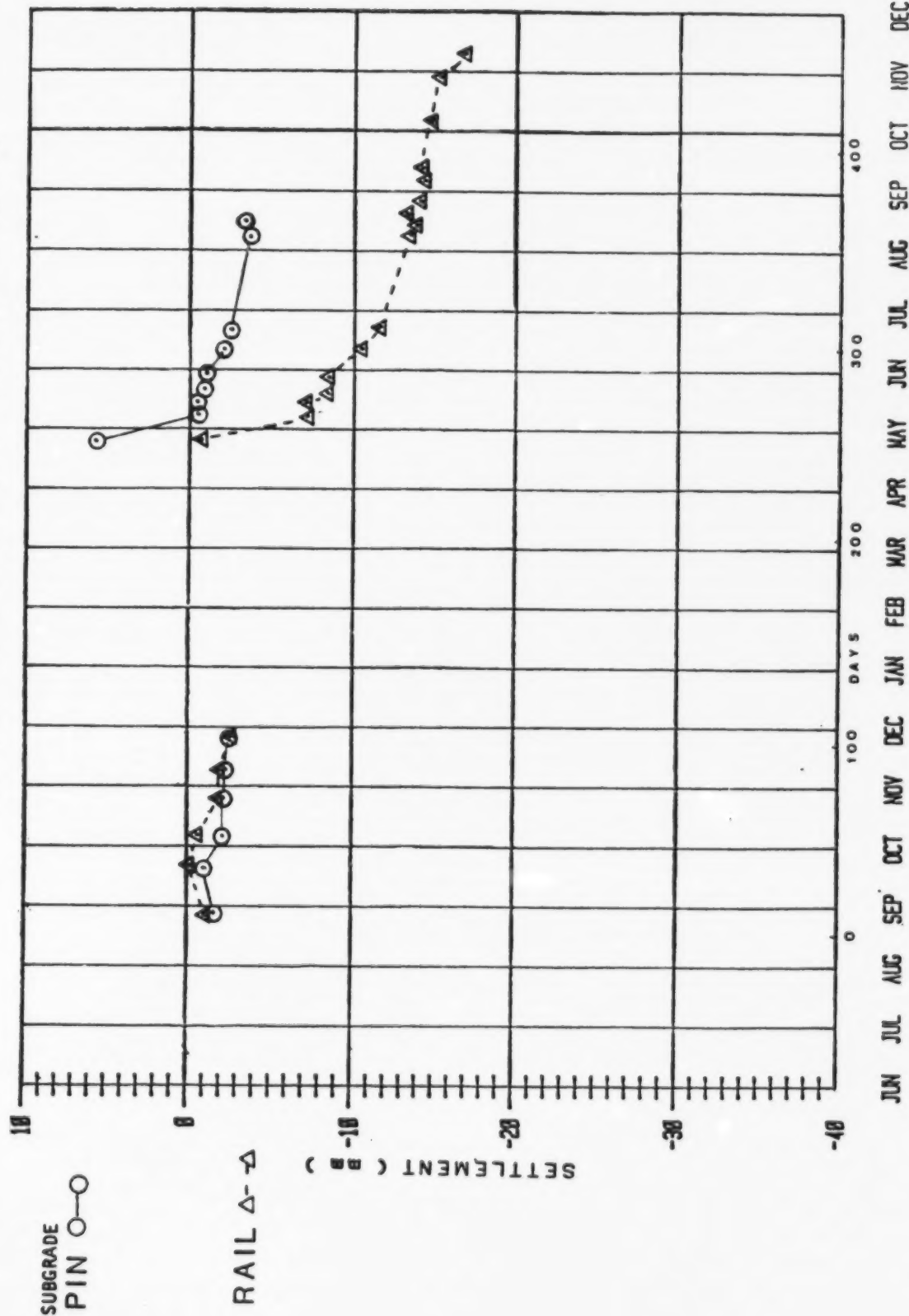
RAIL Δ



APR MAY JUN JUL AUG SEP OCT

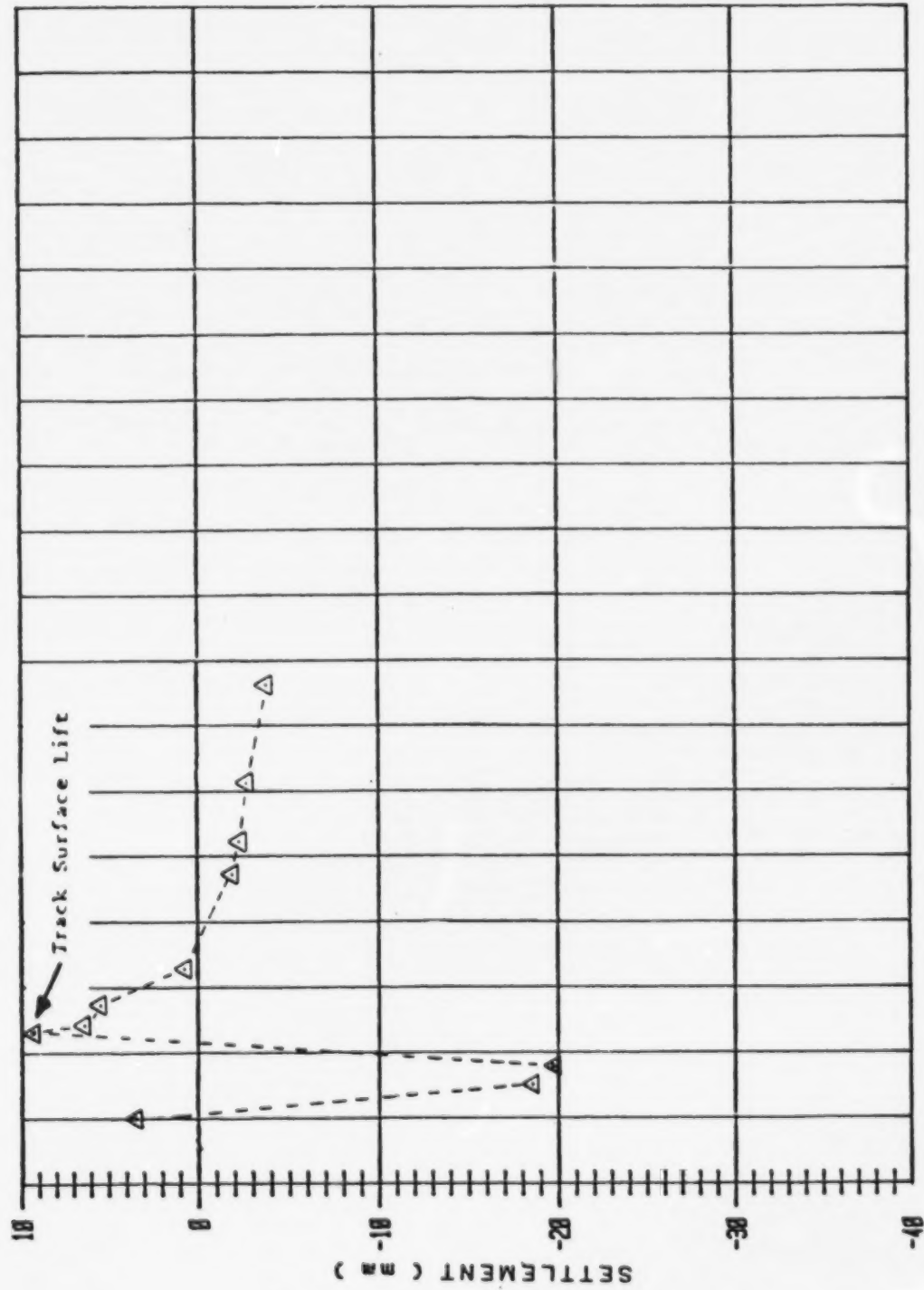
SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18-1K



SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18



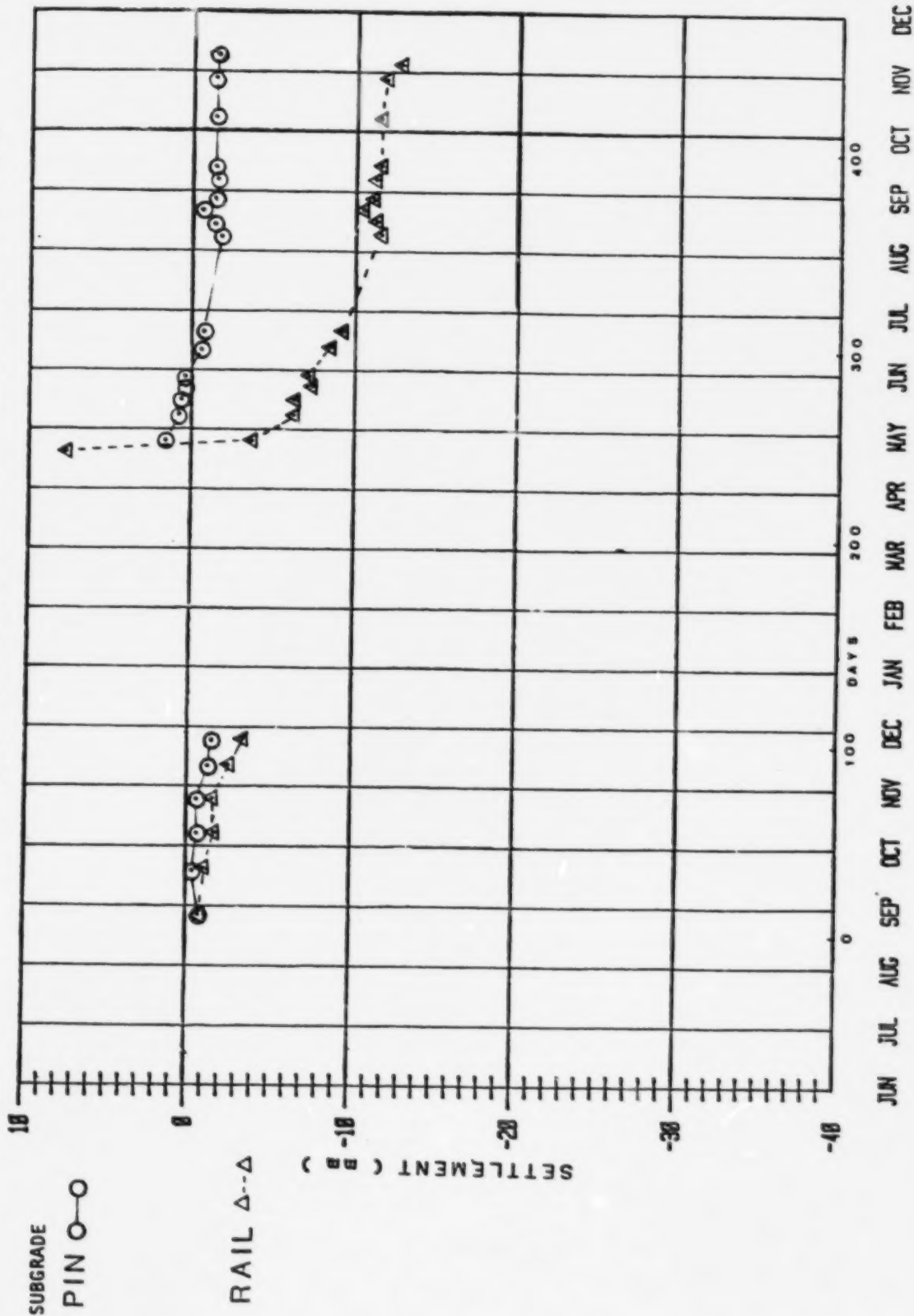
1K
SUBGRADE
PINO

RAIL Δ

APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

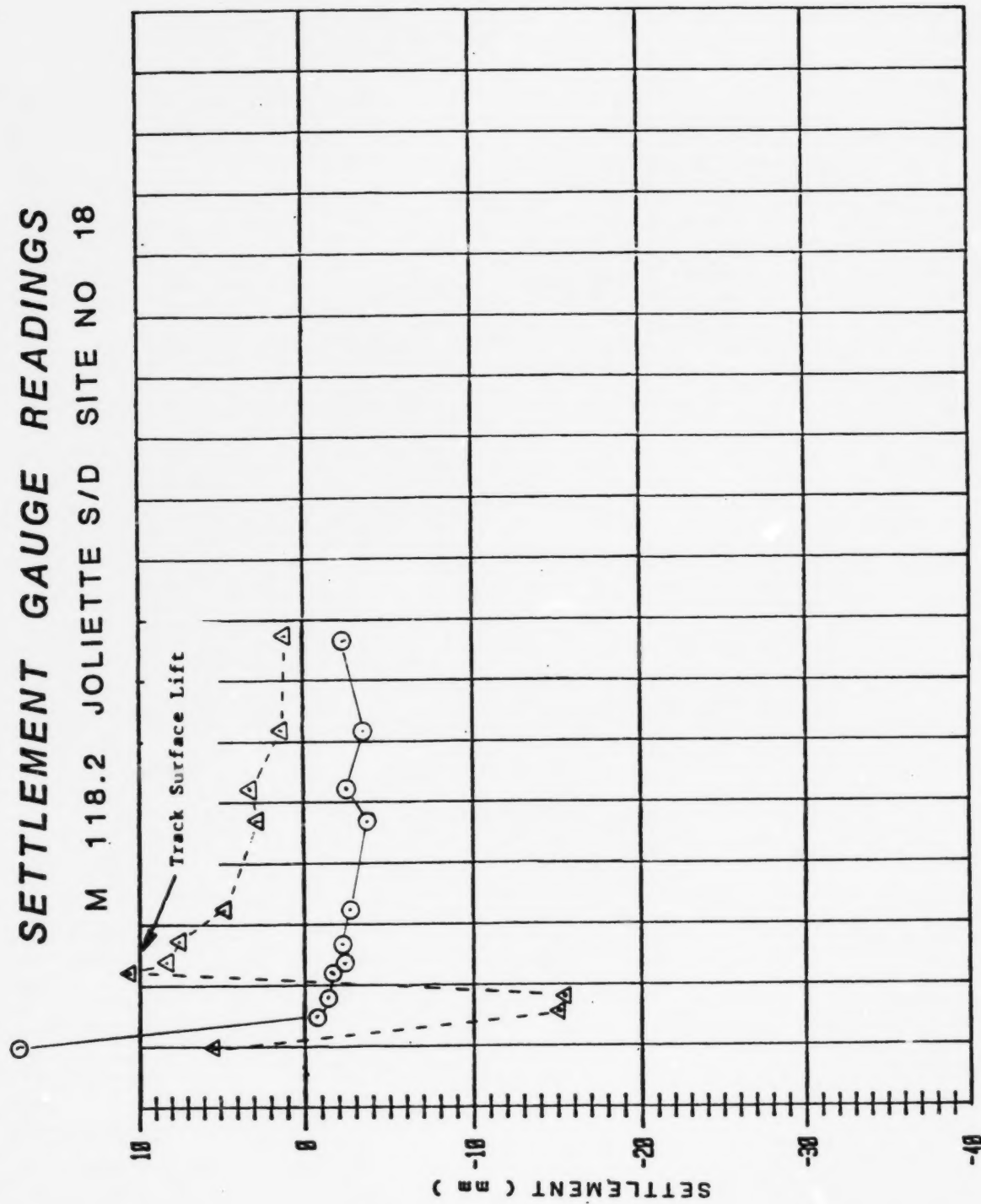
SETTLEMENT GAUGE READINGS

M. 118.2 JOLIETTE S/D SITE NO 18-1 M



SETTLEMENT GAUGE READINGS

M 118.2 JOLIETTE S/D SITE NO 18



APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

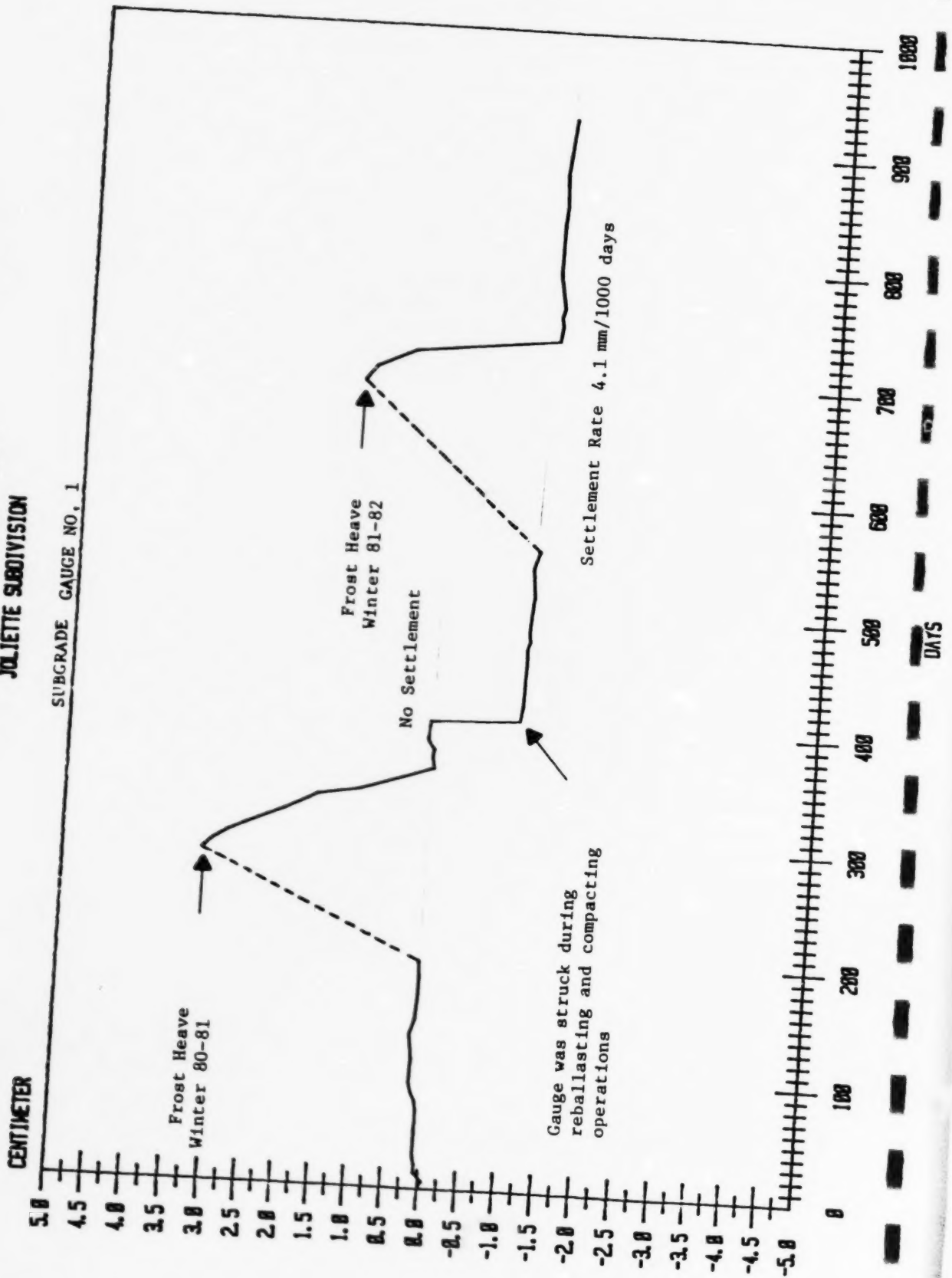
1982 MONTHS

APPENDIX B

Subgrade and Rail Settlements Site No. 19, Mile 123.4 Joliet
Subdivision Period 1980-1982.

SETTLEMENT GAUGES

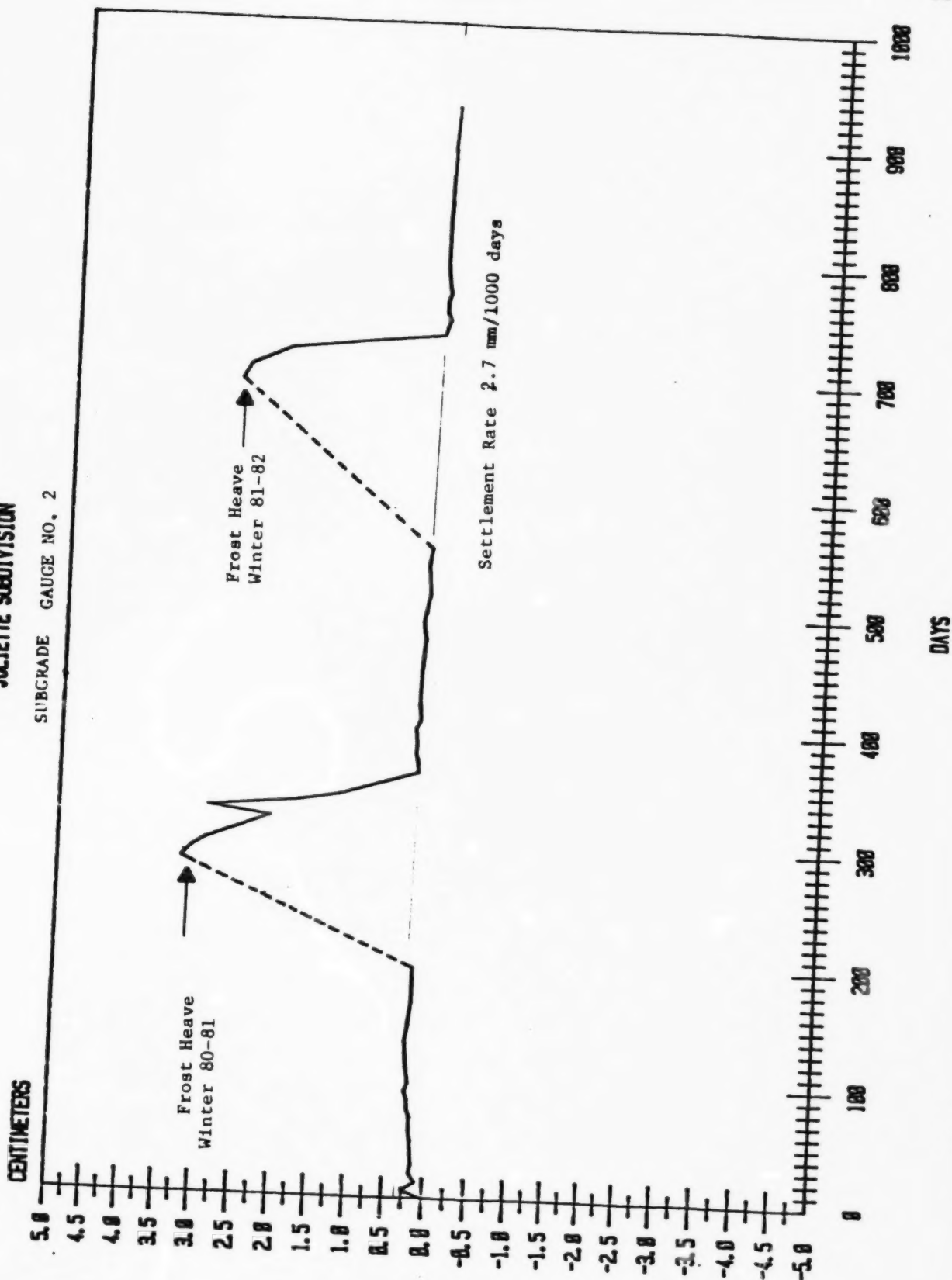
JOLIETTE SUBDIVISION



SETTLEMENT GAUGES

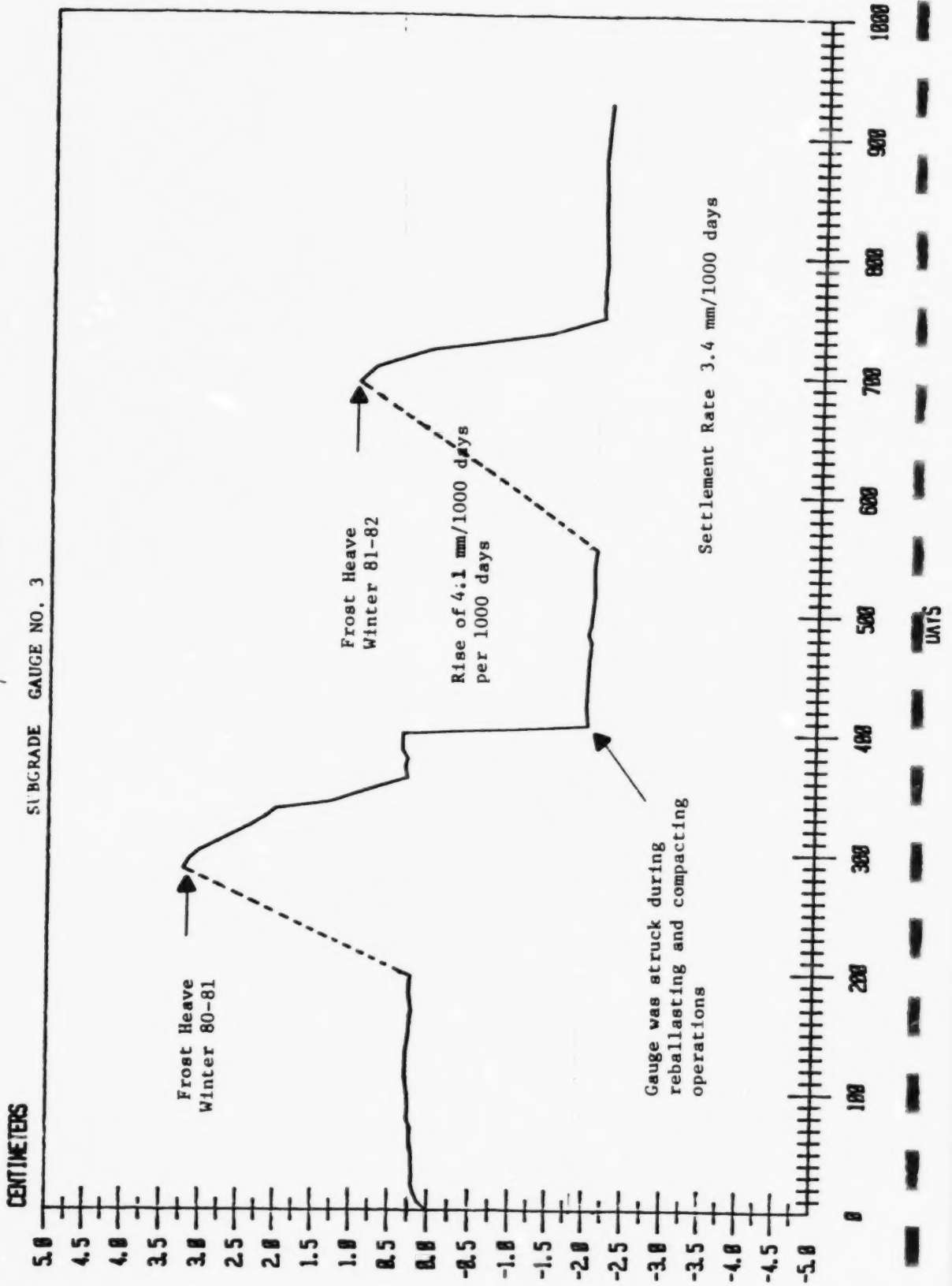
JULIETTE SUBDIVISION

SUBGRADE GAUGE NO. 2



SETTLEMENT GAUGES

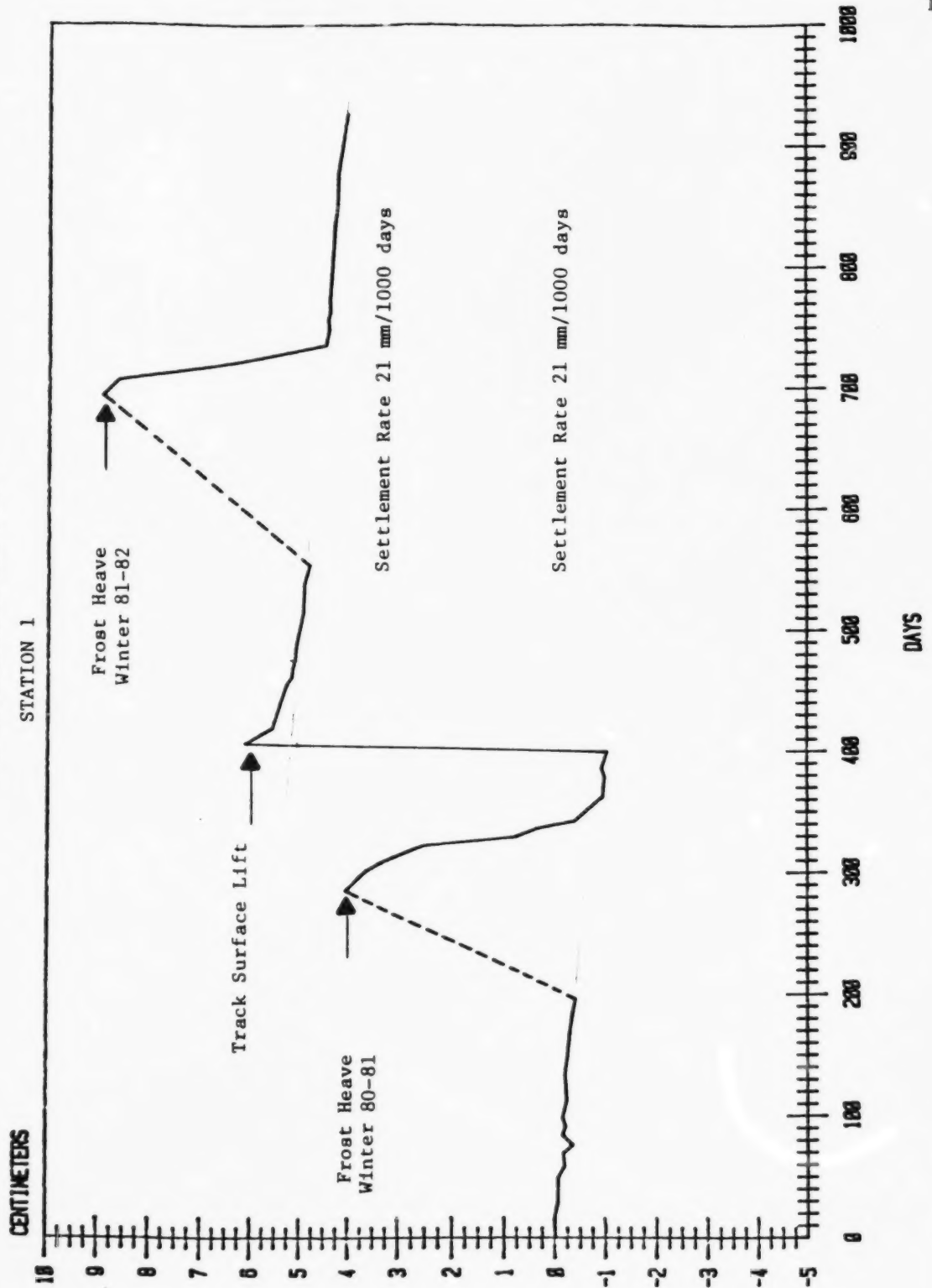
JOLLETTE SUBDIVISION



RAIL SETTLEMENT

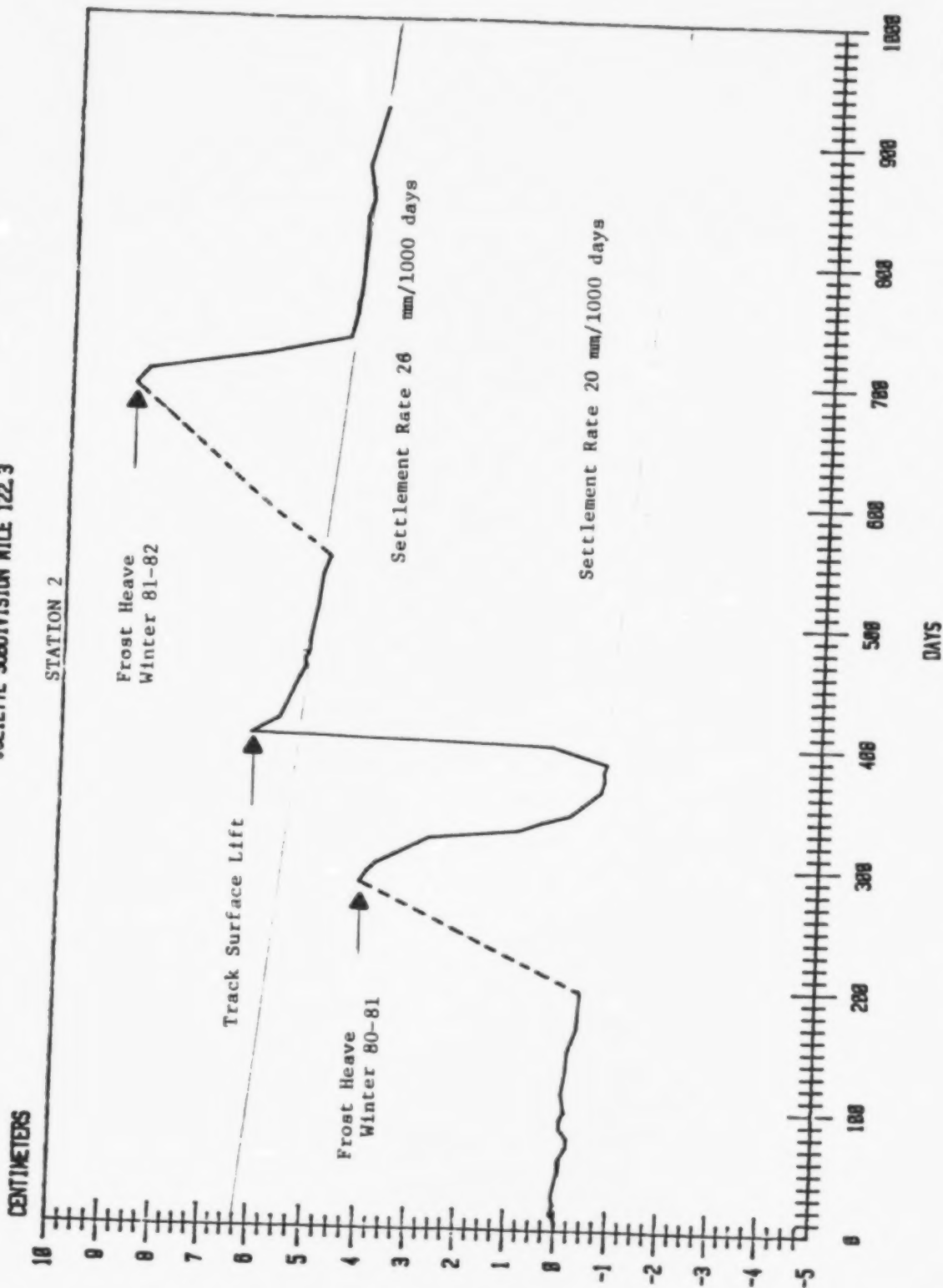
JOLIETTE SUBDIVISION MILE 122.3

STATION 1



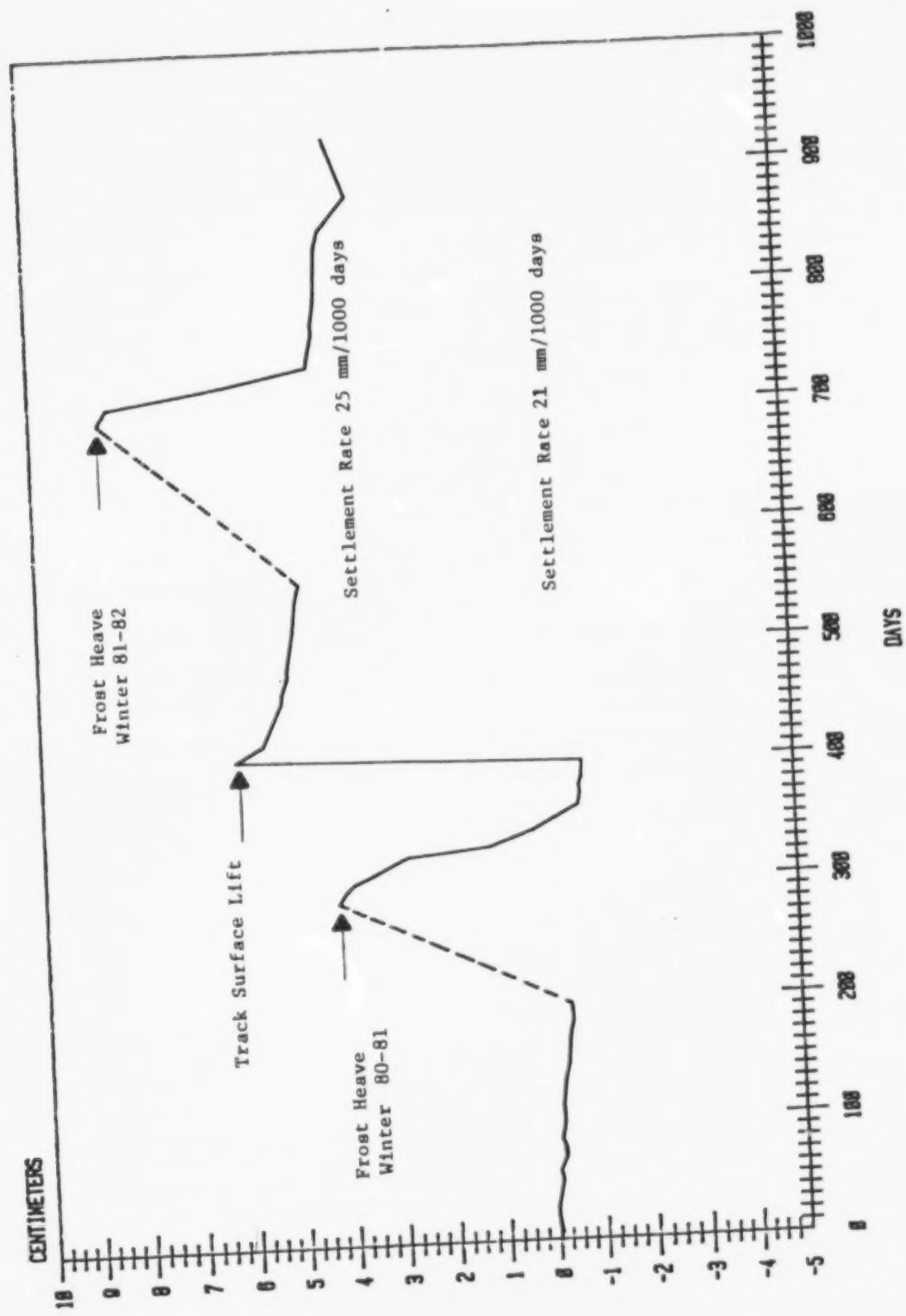
RAIL SETTLEMENT

JOLIETTE SUBDIVISION MILE 122.3



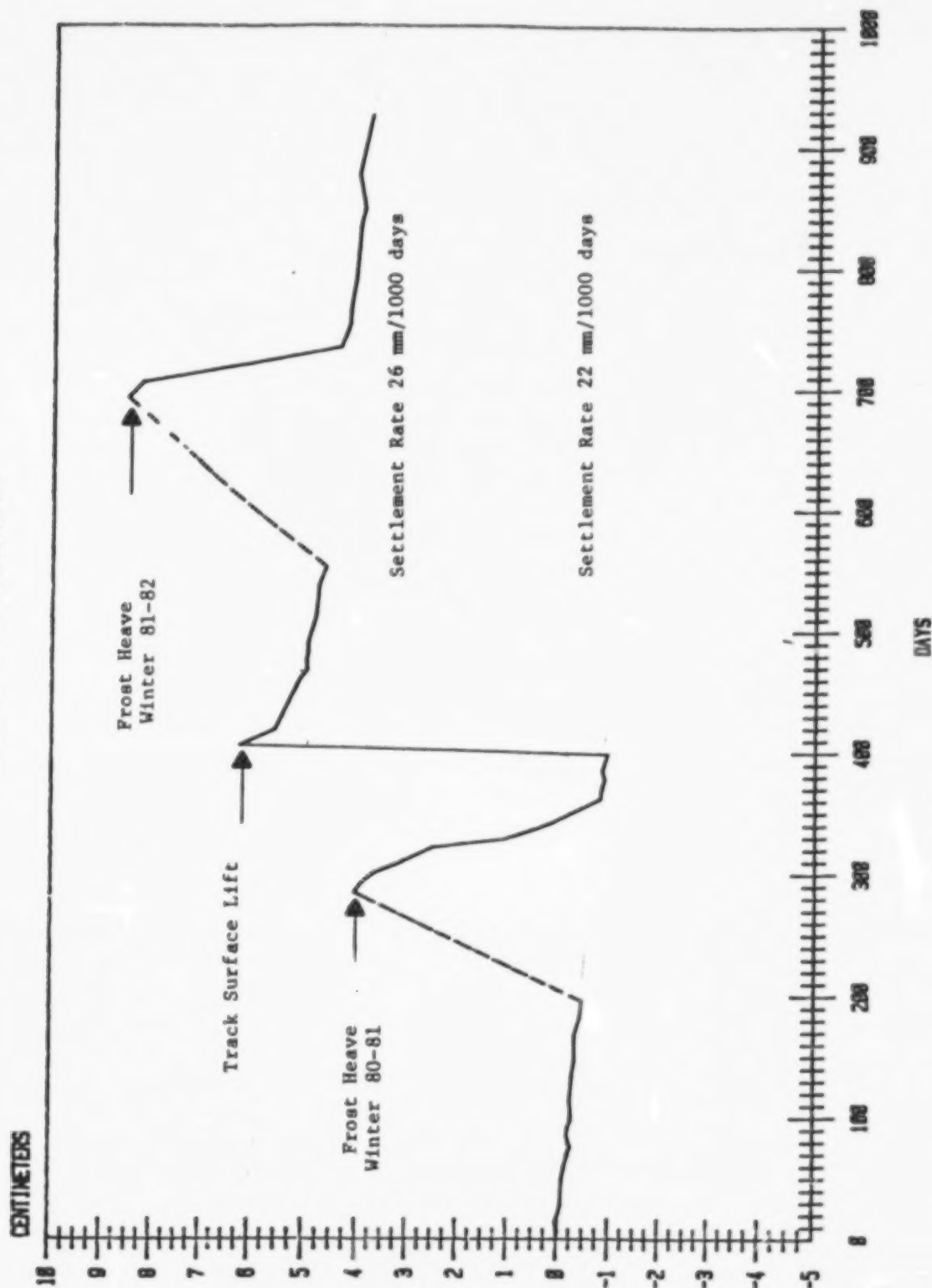
RAIL SETTLEMENT

STATION 3 MILE 122.3 JOLIETTE S/D



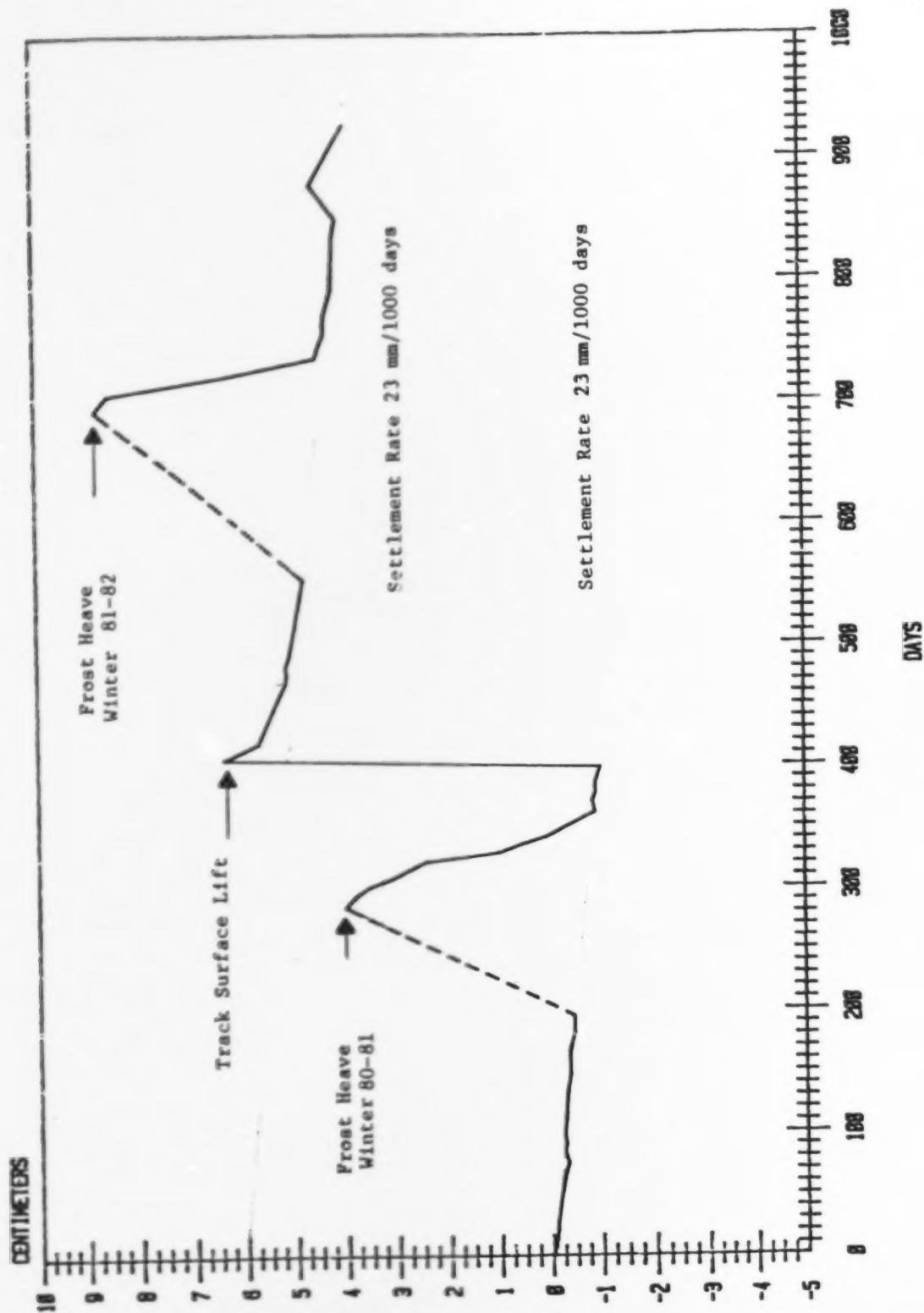
RAIL SETTLEMENT

STATION 4 MILE 122.3 JOLIETTE S/D



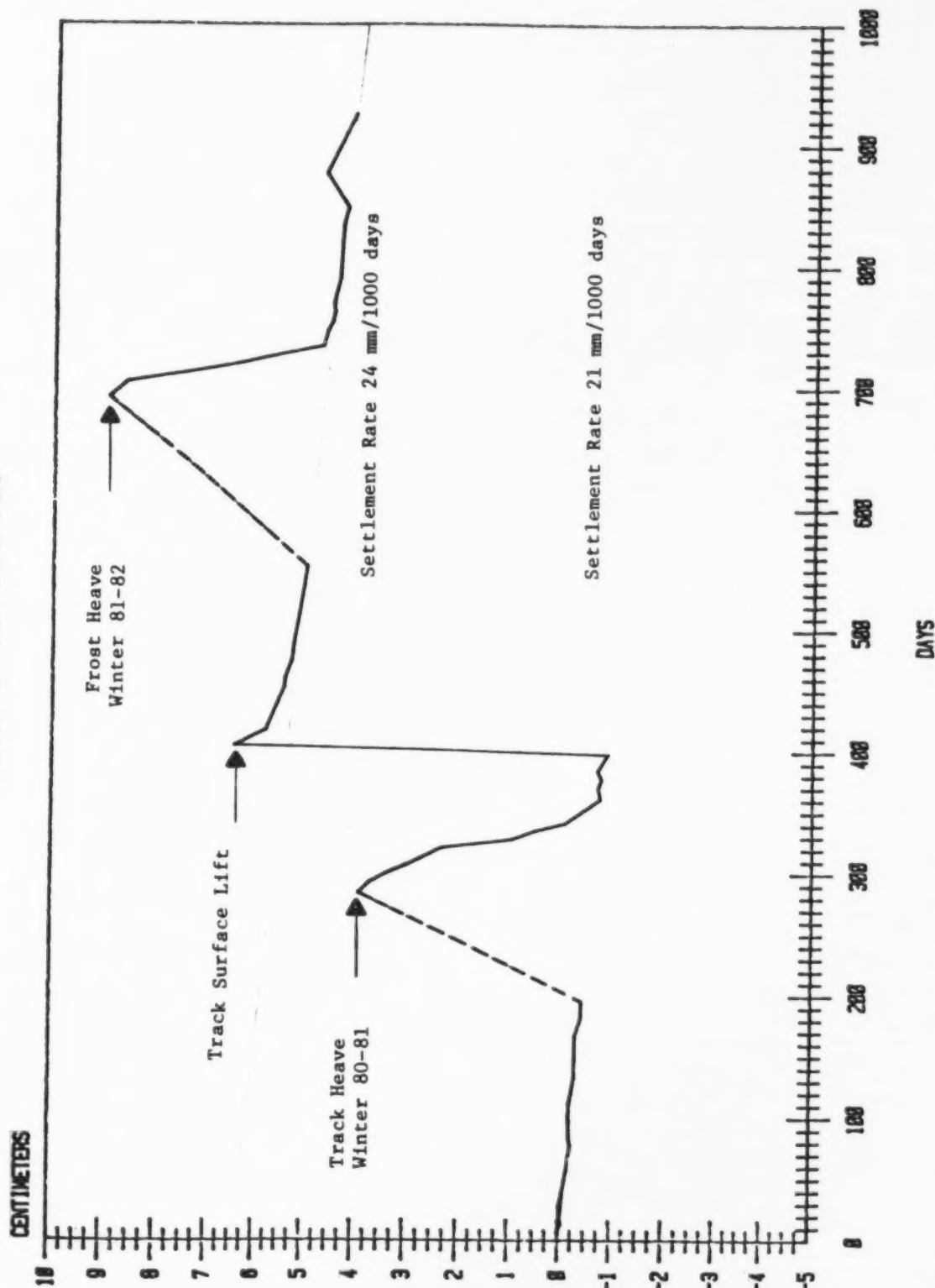
RAIL SETTLEMENT

STATION 5 MILE 122.3 JOLIETTE S/D



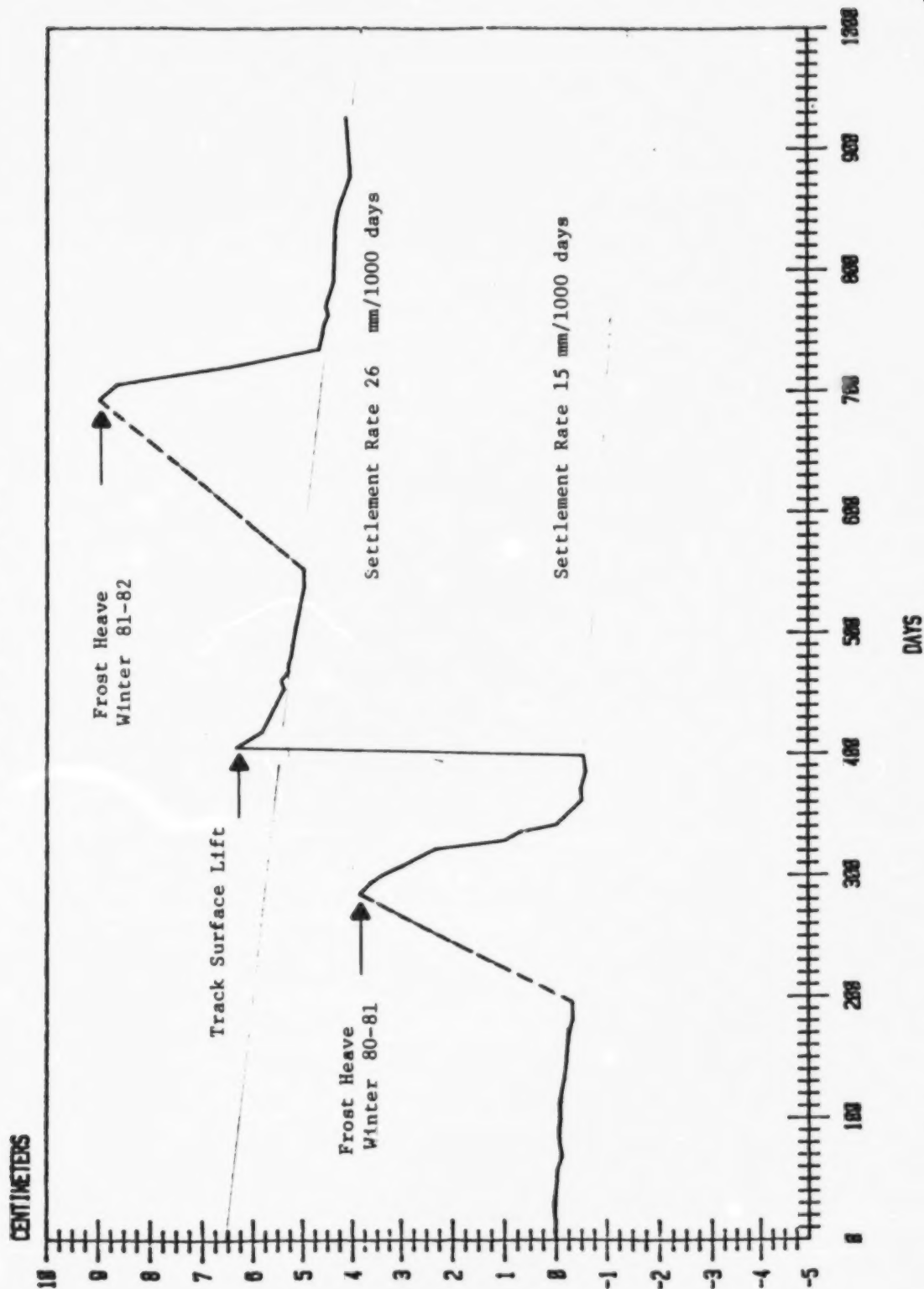
RAIL SETTLEMENT

STATION 6 MILE 122.3 JULIETTE S/D



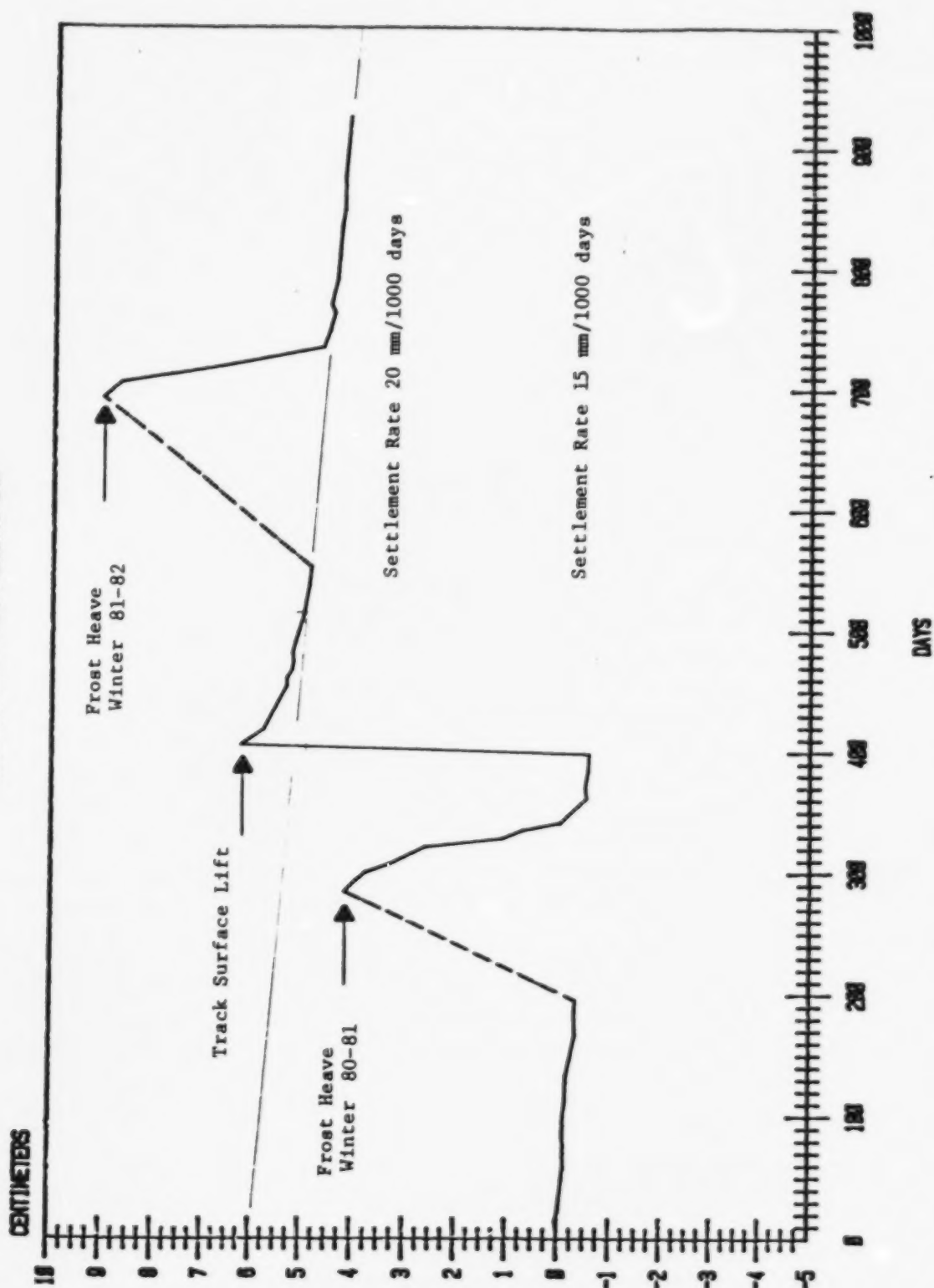
RAIL SETTLEMENT

STATION 7 MILE 122.3 JOLIETTE S/O



RAIL SETTLEMENT

STATION 8 MILE 122.3 JOLIETTE S/D



APPENDIX C

Subgrade Settlements - Western Canada

Site No. 11 - Mile 257.8 Wainwright Subdivision

Site No. 14 - Mile 9.67 Edson Subdivision

Site No. 15 - Mile 38.7 Edson Subdivision

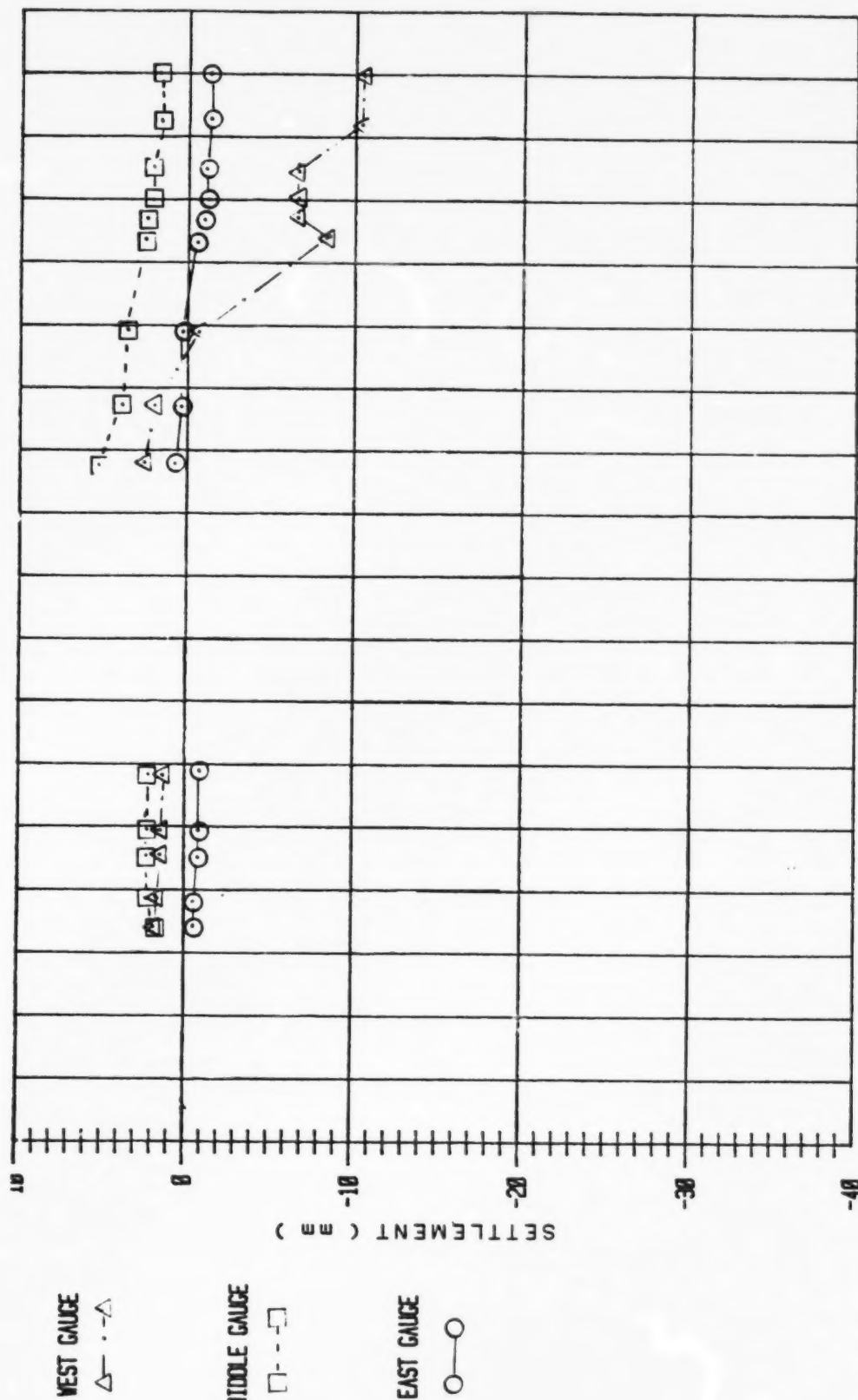
Site No. 16 - Mile 123.4 Vegreville Subdivision

Site No. 17 - Calder Yard Bypass, Edmonton

Period 1980 - 1982

SETTLEMENT GAUGE READINGS

M 257.8 WAINWRIGHT S/D SITE NO 11

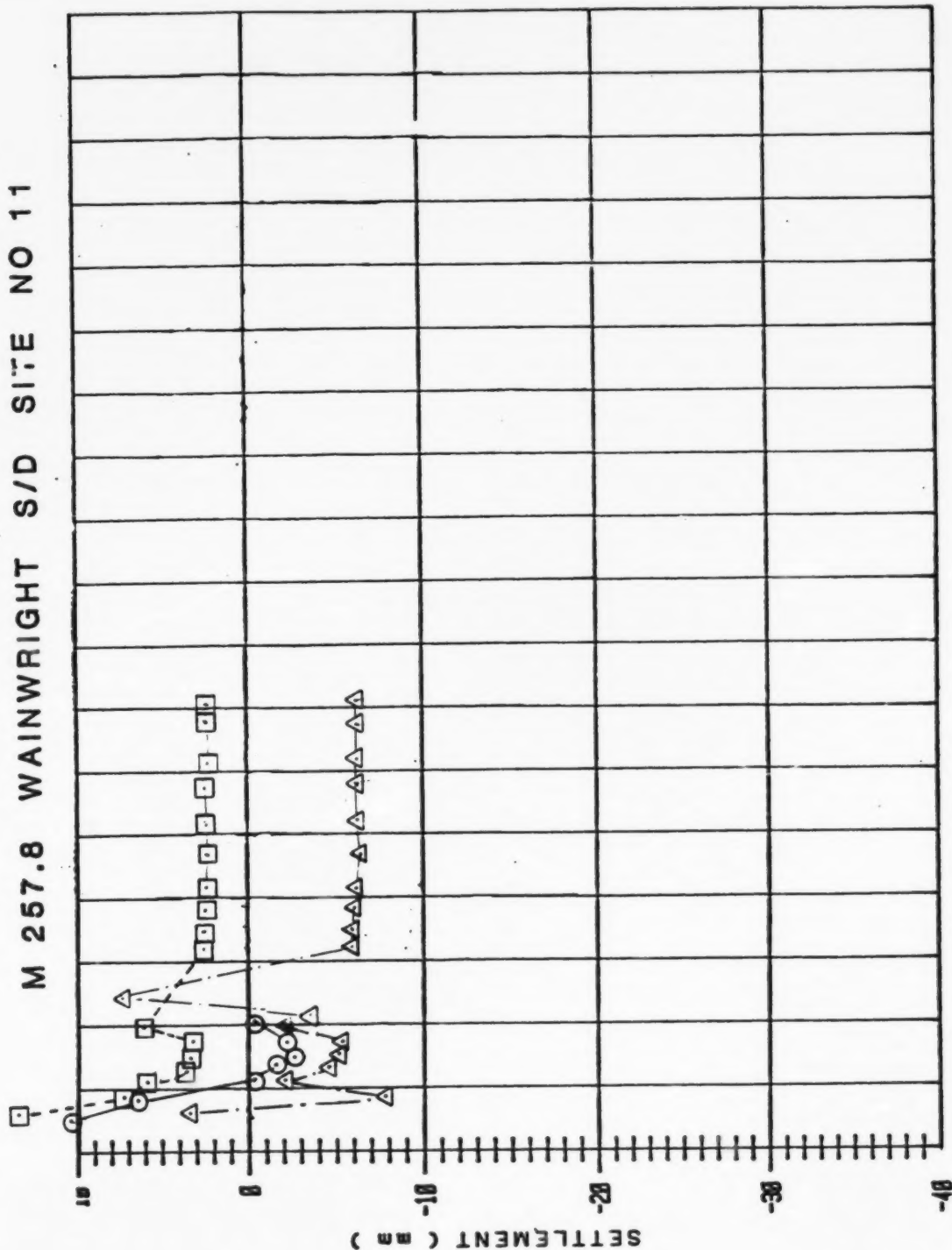


JUN JUL AUG SEP OCT NOV DEC

MONTHS (1981 - 1981)

SETTLEMENT GAUGE READINGS

M 257.8 WAINWRIGHT S/D SITE NO 11

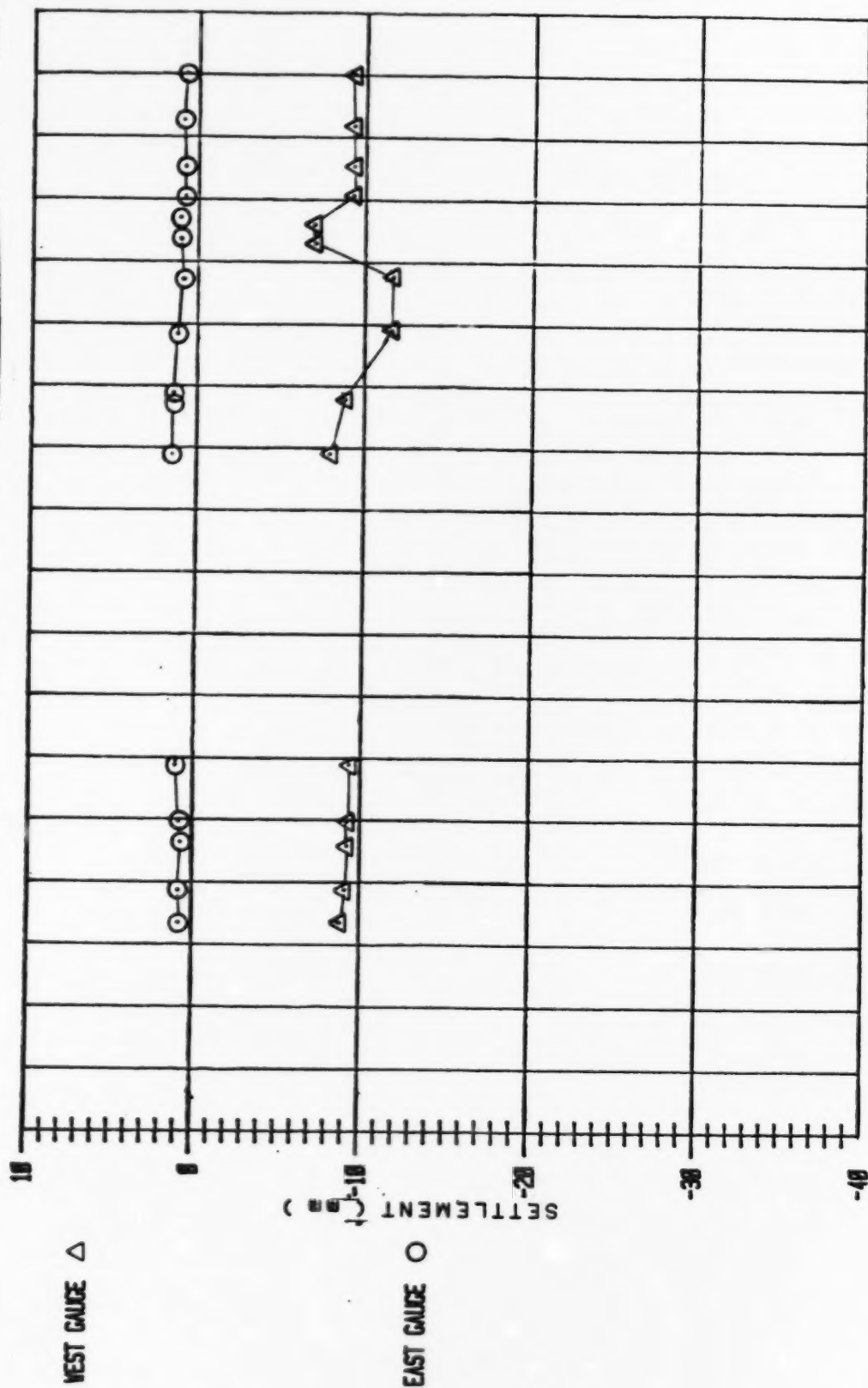


APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

1982 MONTHS

SETTLEMENT GAUGE READINGS

M 9.67 EDSON S/D SITE NO 14

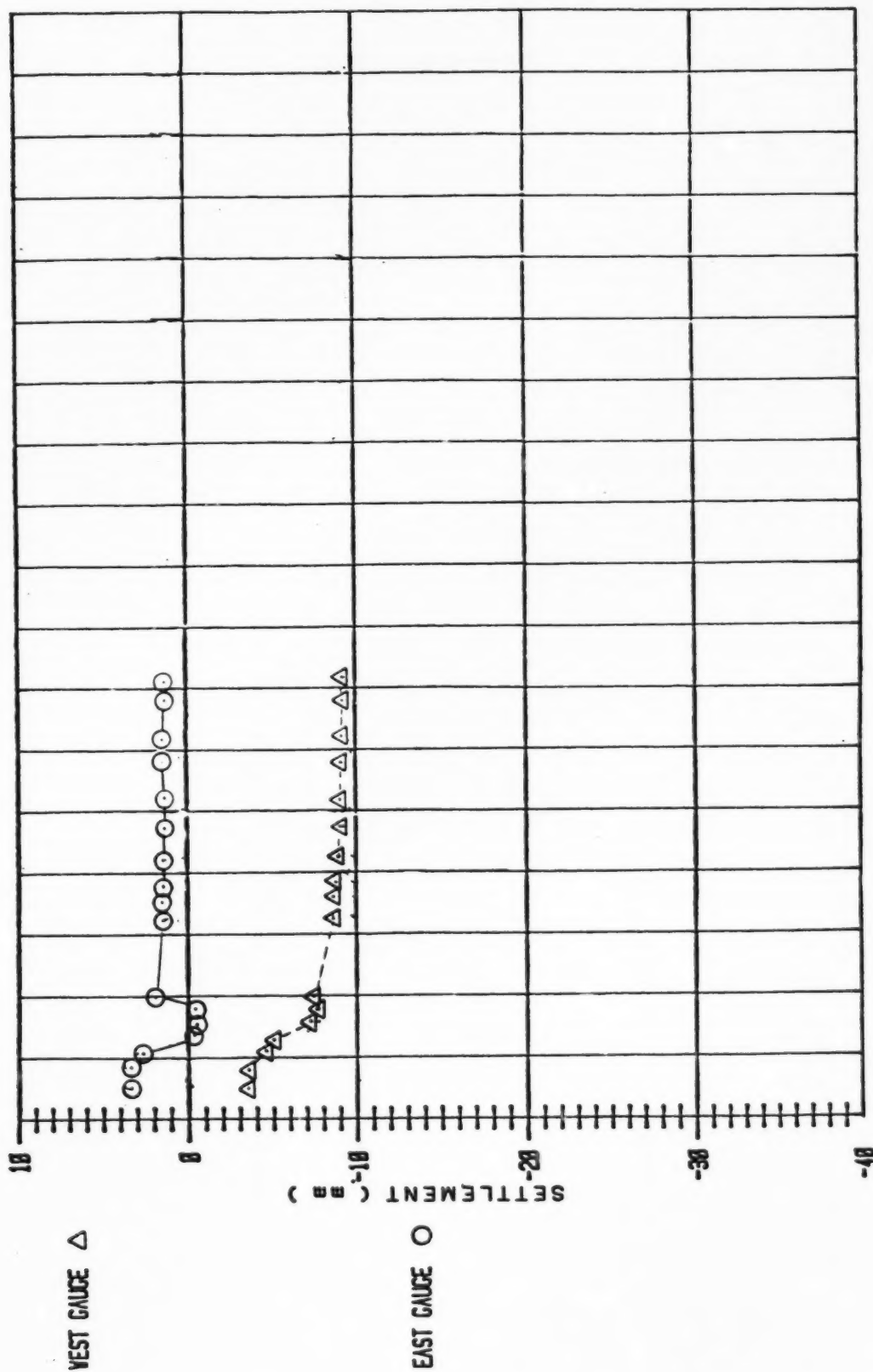


JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC

1967 (1967-1968)

SETTLEMENT GAUGE READINGS

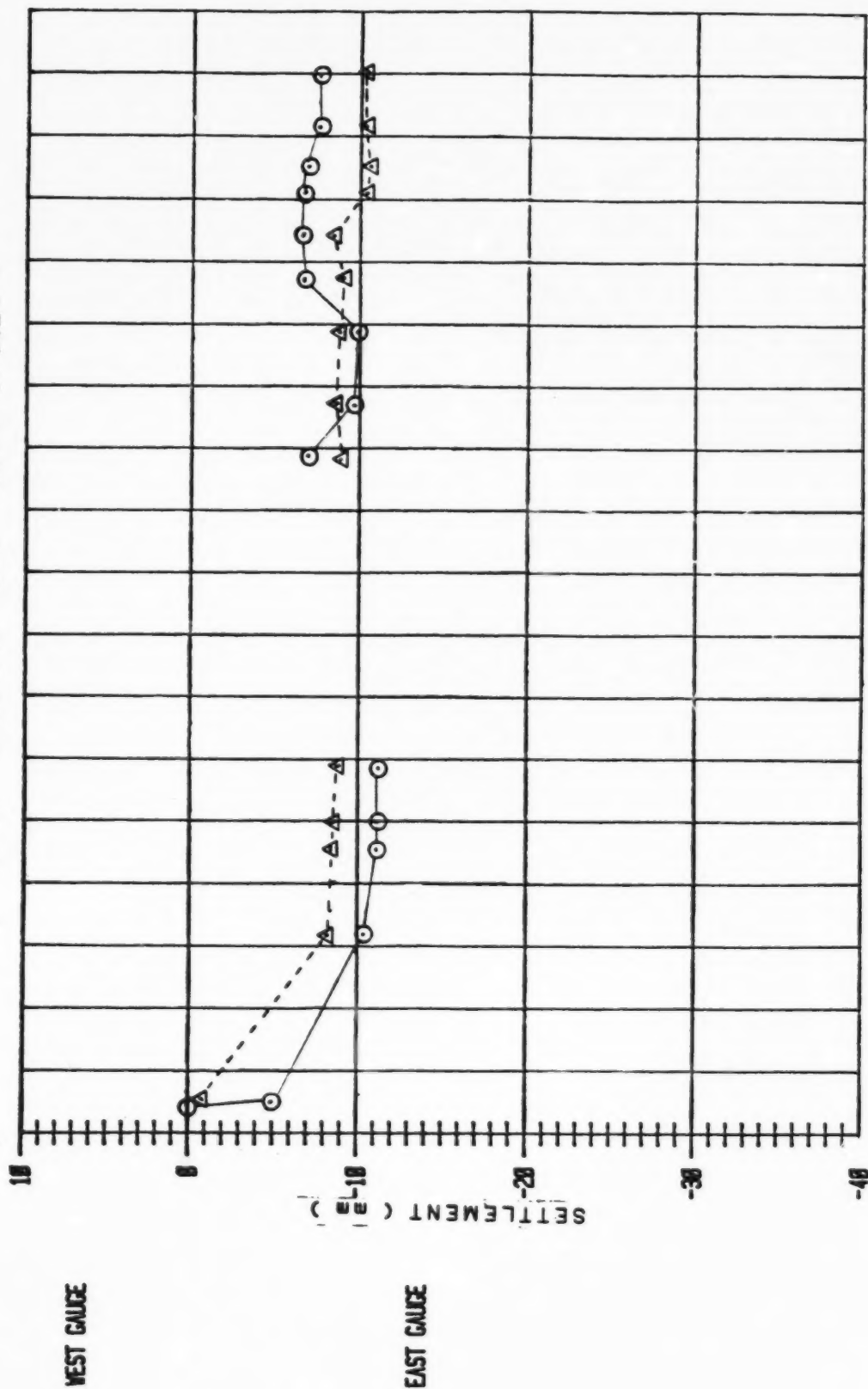
M 9.67 EDSON S/D SITE NO 14



APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

SETTLEMENT GAUGE READINGS

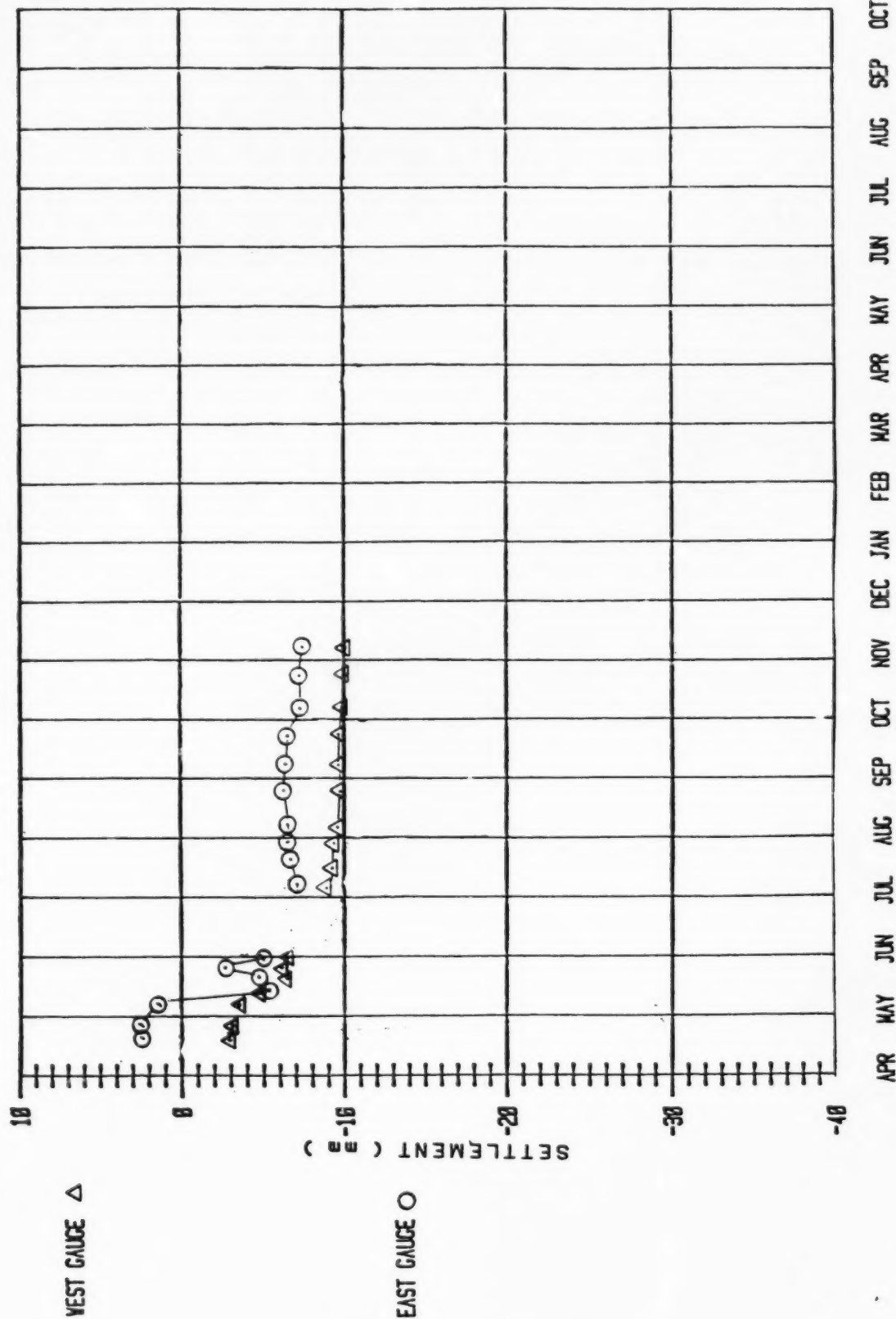
M 38.7 EDSON S/D SITE NO 15



JUN JUL AUG SEP OCT NOV DEC

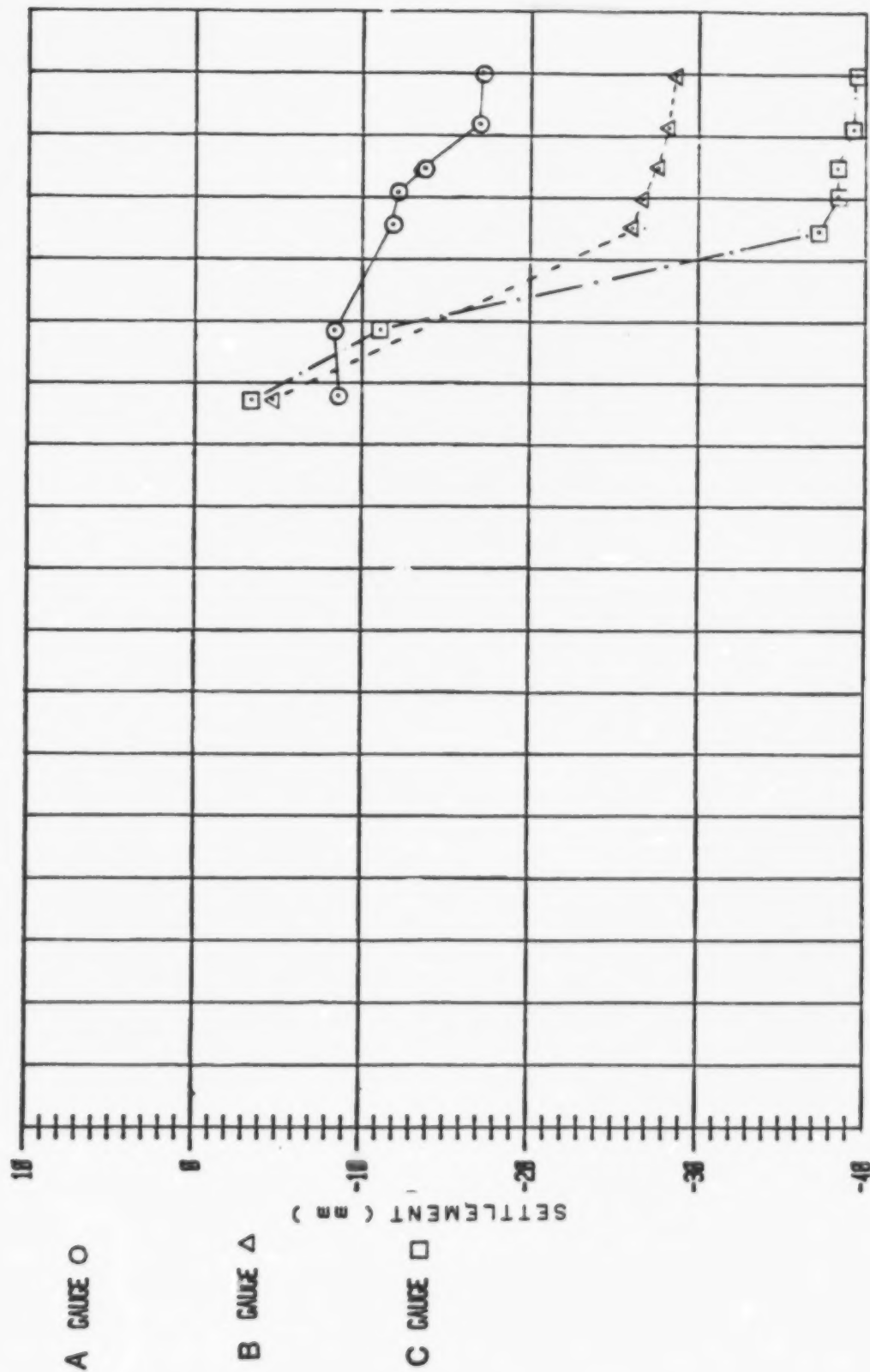
SETTLEMENT GAUGE READINGS

M 38.7 EDSON S/D SITE NO 15



SETTLEMENT GAUGE READINGS

M 123.4 VEGREVILLE S/D SITE NO16

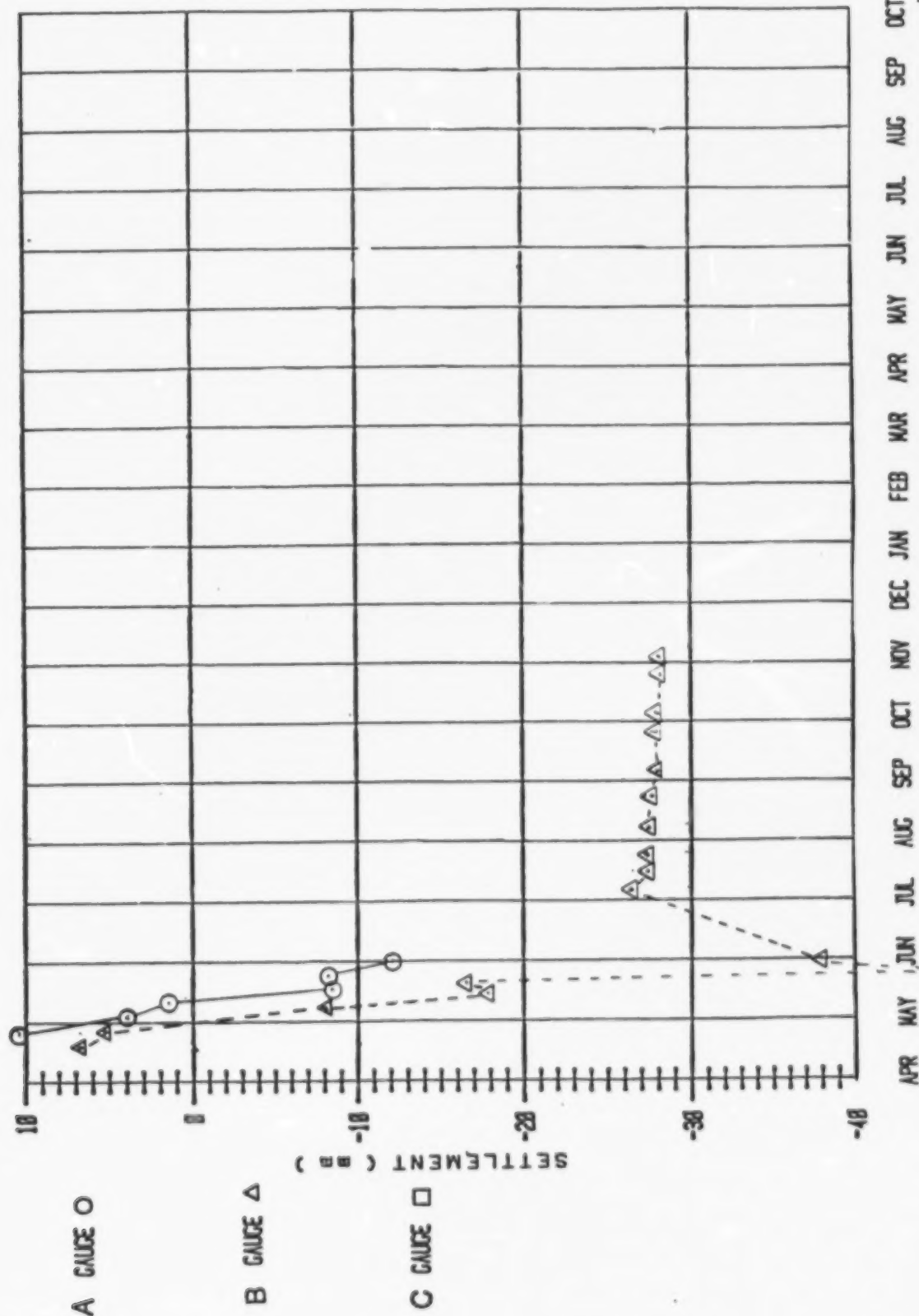


JUN JUL AUG SEP OCT NOV DEC JUN JUL AUG SEP OCT NOV DEC

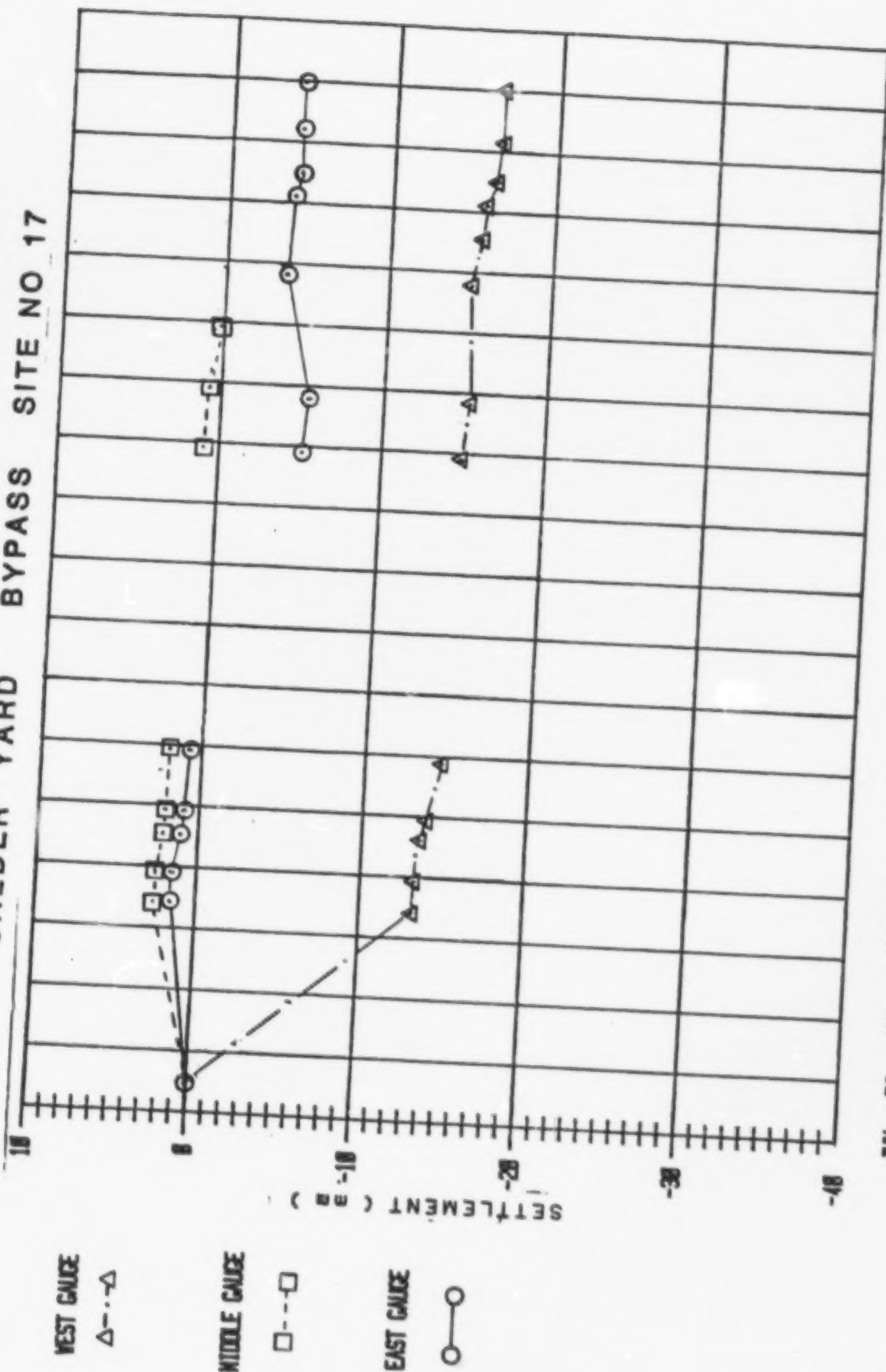
NOV 1981

SETTLEMENT. GAUGE READINGS

M 123.4 VEGREVILLE S/D SITE NO16



SETTLEMENT GAUGE READINGS CALDER YARD BYPASS SITE NO 17

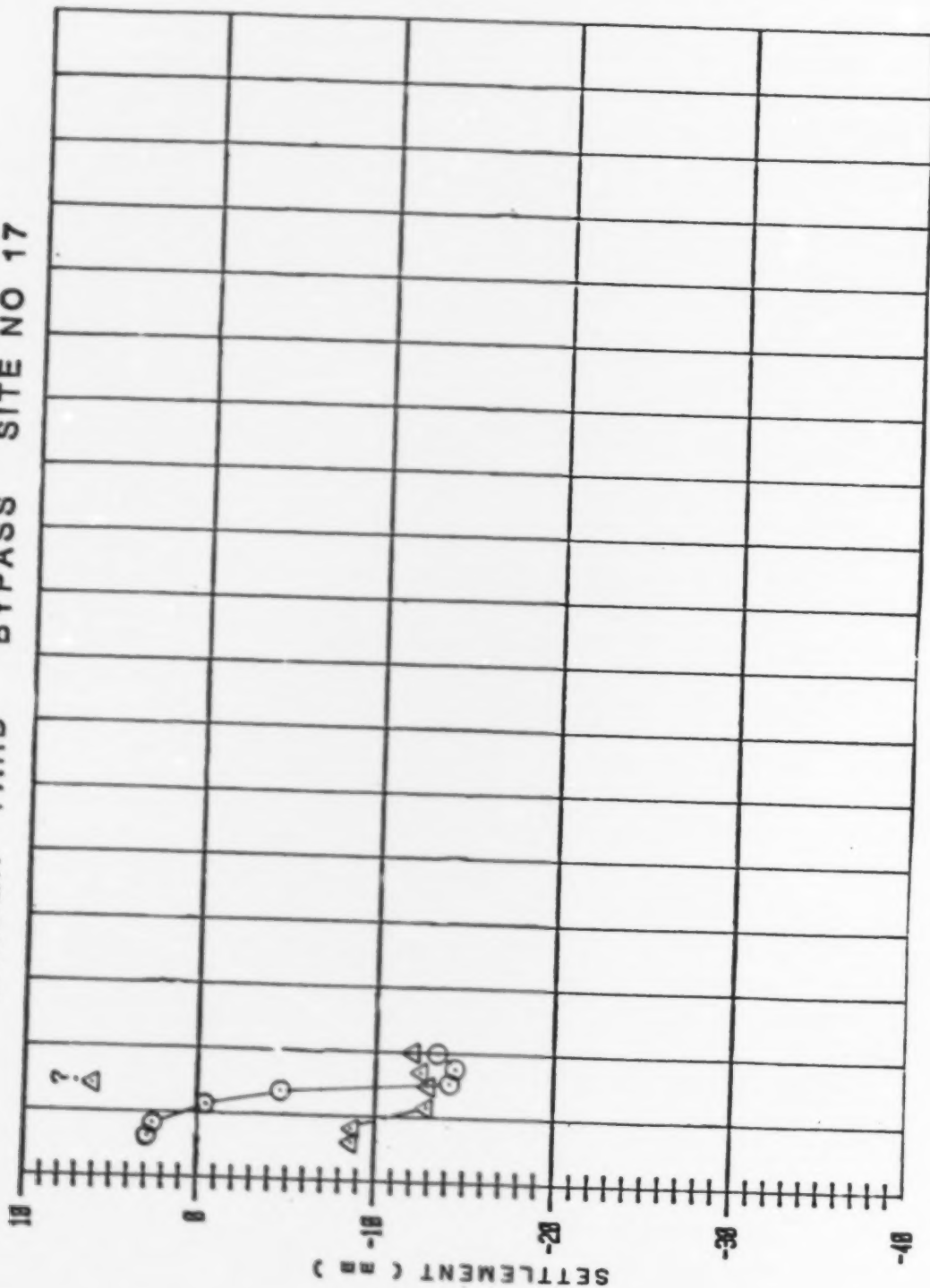


JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC
MONTHS (1980 - 1981)

NOV 1981

SETTLEMENT GAUGE READINGS

CALDER YARD BYPASS SITE NO 17



WEST GAUGE

△-△

MIDDLE GAUGE

□-□

EAST GAUGE

○-○

APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT

1982

MONTHS



APPENDIX D

CHARACTERIZATION OF RAILROAD TRACK VERTICAL LOADING

CHARACTERIZATION OF RAILROAD TRACK VERTICAL LOADING

Railroads keep records of traffic on all territories in terms of million gross ton-miles per mile of track. For any specific point in track, the traffic may be simply defined in terms of million gross tons (MGT).

The tonnage is based upon the static total weight (tare plus payload) of the vehicles carried over the territory. By definition, freight vehicles have been classified into loaded weight groups:

100-ton car - 263,000 lbs. or 131.5 tons maximum on rail
 70-ton car = 220,000 lbs. or 110 tons maximum on rail
 50-ton car = 177,000 lbs. or 88.5 tons maximum on rail

It is to be noted that maximum weights on rail have been specified. A great many of these vehicles are moved with partial loads and there are many return movements with empty vehicles. All movements accumulate tonnage. Wheel loads applied to the track are dynamic and they range all the way from 0.5 to 25 tons (and even greater in some cases). The tonnage over a territory is thus comprised of a broad spectrum of loading.

It is the purpose of this resumé to categorize railroad track vertical loading into a form which is useful for correlation with:

- (a) Rate of track surface deterioration and maintenance requirements.
- (b) Dynamic tests conducted in the laboratory.

To put things in perspective, consider two territories A and B, each with identical tonnages (MGT), but with territory A having a high concentration of 100-ton cars in unit train operations, and territory B having a general mix of lighter freight and passenger train operations. Territory A would have higher wheel loadings but fewer cycles of loading than territory B. Which track would require more maintenance if roadbed conditions were similar?

The tonnage over the territory is important.
 The magnitude of axle loads is important.
 The number of axle loadings is important.

Many peak vertical dynamic wheel loads have been measured on six subdivisions across the CN system and wheel loading probability curves have been drawn (see Reference No. 1). The data has now been reduced to show CN traffic in terms of percentage of total axles and percentage of gross tonnage within given axle load ranges. This information has been tabulated in Figure No. 1. There are significant differences in axle loadings from subdivision to subdivision. It is worth noting that tonnages within given load ranges were derived from the measured dynamic wheel loads, and it is not known how well the values correlate with records from static load data. It has simply been assumed that dynamic loads vary with normal distribution about the mean static load.

CN TRAFFIC IN TERMS OF PERCENTAGE OF AXLES AND PERCENTAGE OF TONNAGE IN GIVEN AXLE LOAD RANGES

AXLE LOAD, TONS		KINGSTON SUB.				YALE SUB.				RIVERS SUB.				SPRAGUE SUB.				NEWMARKET SUB.				STRATHROY SUB.				JOLIETTE SUB.				CN SYSTEM	
		% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS	% AXLES	% TONS						
RANGE	AVG.																														
0-5	4	.062	.015	.472	.093	1.14	.27	.178	.043	0	0	.086	.02	1.00	.16	.46	.10														
5-10	7.5	28.88	12.65	25.14	9.27	32.75	14.47	48.15	21.95	26.16	8.54	27.92	12.39	38.5	18.97	30.51	12.84														
10-15	12.5	21.91	16.00	10.65	6.55	13.16	9.69	2.67	2.03	22.81	12.41	22.94	16.97	11.5	9.44	15.86	11.13														
15-20	17.5	16.00	16.36	14.15	12.18	16.57	17.08	9.54	10.15	3.02	2.30	17.20	17.82	19.5	22.42	15.08	14.81														
20-25	22.5	11.81	15.52	13.01	14.39	16.78	22.24	19.30	26.39	4.04	3.96	12.00	15.98	18.0	26.61	13.84	17.48														
25-30	27.5	8.14	13.08	11.00	14.87	7.58	12.28	7.55	12.62	7.89	9.44	8.71	14.18	7.3	13.19	8.60	13.27														
30-35	32.5	8.81	16.73	18.57	29.68	8.72	16.63	6.50	12.84	6.17	8.73	7.42	14.27	3.55	7.58	10.25	18.70														
35-40	37.5	4.15	9.09	6.75	12.44	3.19	7.05	5.83	13.29	7.89	12.88	3.31	7.35	.59	1.45	4.51	9.49														
40-45	42.5	.215	.53	.25	.52	.092	.23	.272	.70	17.52	32.41	.37	.93	.06	.17	.75	1.79														
45-50	47.5	.008	.02	.001	.002	.002	.01	0	0	4.30	8.89	.028	.03	0	0	.14	.37														
>50	51	.001	.003	0	0	0	0	0	0	.19	.42	.004	.01			.007	.02														
TOTAL AXLES, NO.		100,057		85,047		118,572		33,827		13,041		72,276		5,024		422,820															
TOTAL TONS, MGT		1,713,017		1,729,834		2,013,389		0,556,728		0,299,605		1,221,364				7,533,936															
NO. AXLES/MGT		58,410		49,165		58,892		60,760		43,527		59,176				56,122															

(1) SMALL SAMPLE SIZE, NOT REPRESENTATIVE OF GENERAL TRAFFIC ON SUBDIVISION.

(2) SUMMATION OF TRAFFIC FROM SEVEN LISTED SUBDIVISIONS.

FIGURE NO. 1

While the information in Figure No. 1 pretty well defines the track vertical loading spectra, the data has been reduced to a more usable form in Figure No. 2. The axle loading has been divided into three ranges.

0-20 Tons Light	The relatively light axle loads in this range may account for a significant proportion of total axles over a territory, but their effect on deterioration rate of track may be quite small. At this point in time, it is felt that all of these loadings may be safely ignored with respect to roadbed design criteria.
20-30 Tons Medium	The intermediate axle loads may or may not cause significant track deterioration depending upon the traffic mix on a territory. These loads <u>should</u> be taken into account. While the percentages of loads in the 20 to 25 and 25 to 30 ton ranges were different on all territories, for the sake of simplification it may be assumed that the mean axle load of the 20 to 30 ton range was 25 tons.
>30 Tons Heavy	This heavy loading range basically encompasses the loadings imposed by freight locomotives and loaded 100-ton cars. The mean axle dynamic load for this range has been determined as 34.6 tons.

In Figure No. 2 (b), the numbers of axles per year in the three load ranges have been determined for specific subdivisions. On the Kingston Subdivision, with 22.4 MGT and 1.3 million axles per year, 0.87 million axles were in the light load range (which may be ignored), 0.26 million axles were in the medium load range, and 0.17 million axles were in the heavy load range. On the Yale Subdivision, with 42.5 MGT and 2.08 million axles per year, 1.05 million axles may be ignored, 0.5 million were in the medium load range, and 0.53 million axles were in the heavy load range. There were three times as many heavy axles on the Yale Subdivision as there were on the Kingston Subdivision. On the Rivers Subdivision there were 0.42 million more total axles than there were on the Yale Subdivision, but there were only 57 percent as many heavy axles. Numbers of this nature should have a direct bearing on the roadbed design/maintenance requirements.

Track vertical loading spectra have only been determined on a limited number of subdivisions, and loading varies from territory to territory. The available information may be used to estimate the numbers of axles per year in each load range for any territory from a knowledge of the general traffic mix and the total MGT.

The percentage of total axles in each load range has been plotted against the number of axles per MGT (total) in Figure No. 3. To utilize this graph, it is necessary to know the MGT for a territory and to estimate the percentage of heavy axles operating over that territory. Assume one wished to estimate the number of axles over a territory with 40 MGT and estimated 20% heavy axles. From the graph, it is determined that there would be 53,000 axles per MGT or $53,000 \times 40 = 2,120,000$ total axles per year.

57% of the axles would be light	=	1,208,000 Axles
23% of the axles would be medium	=	488,000 Axles
20% of the axles would be heavy	=	424,000 Axles

(a) CN TRAFFIC IN TERMS OF PERCENTAGE OF AXLES
AND PERCENTAGE OF TONNAGE IN GIVEN LOAD RANGES

Axle Load Range, Tons	Kingston Sub.		Yale Sub.		Rivers Sub.		CN System	
	% Axles	% Tons	% Axles	% Tons	% Axles	% Tons	% Axles	% Tons
0-20	66.85	45.03	50.41	28.09	63.62	41.51	61.91	38.88
20-30	19.95	28.60	24.01	29.26	24.36	34.52	22.44	30.75
>30	13.18	26.37	25.57	42.64	12.00	23.98	15.66	30.37
No. Axles/MGT	58,410		49,165		58,892		56,122	

Note: MGT = Million Gross Tons

The above figures may be used to determine the number of axle loading cycles in each load range per year. The example tonnage figures used below are from the 1980 CN Traffic Density Report.

(b) NUMBER OF AXLES PER YEAR IN GIVEN LOAD RANGES

Axle Load, Tons		Kingston Sub. 22.4 MGT ⁽¹⁾	Yale Sub. 42.4 MGT ⁽²⁾	Rivers Sub. 42.5 MGT ⁽³⁾	CN System 30 MGT ⁽⁴⁾
Range	Mean	No. Axles x10 ⁶	No. Axles x10 ⁶	No. Axles x10 ⁶	No. Axles x10 ⁶
0-20	11.2	0.87	1.05	1.59	1.04
20-30	25	0.26	0.50	0.61	0.38
>30	34.6	0.17 ⁽⁵⁾	0.53	0.30	0.26
Total Axles/Year		1.30	2.08	2.50	1.68

(1) 22.4 MGT Dorval to Coteau

(2) 42.5 MGT Chilliwack to Boston Bar

(3) 42.5 MGT Petrel Jct. to Portage

(4) 30 MGT Assumed

(5) Example Calculation:

No. of axles/year with loads >30 tons

$$\frac{22.4 \text{ MGT}}{\text{Year}} \times .05841 \text{ Million Axles} \times \frac{13.18\% \text{ of Axles in Range}}{100}$$

$$= 0.17 \text{ Million Axles/Year}$$

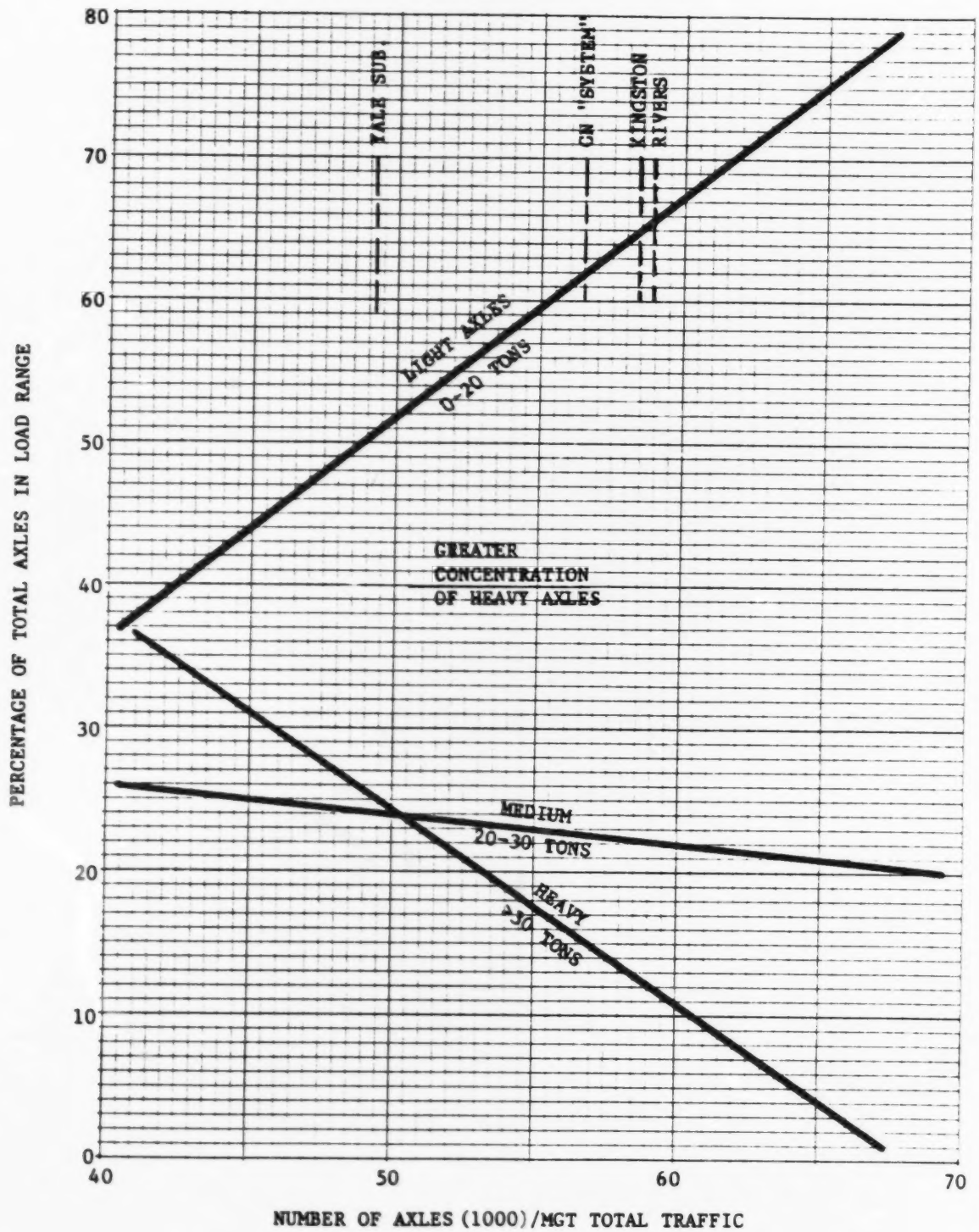
NUMBER OF AXLES/MGT ON CN TRACK

FIGURE NO. 3

The graph in Figure No. 3 is for use on territories with mixed traffic. It would not be applicable for branch line unit train territories. The graph will not provide highly accurate axle counts, but for estimating purposes it is considered reasonably valid.

Track settlement measurements were taken in the Edmonton area on the Edson, Wainwright, and Vegreville Subdivisions. In order to correlate track settlement rates with information derived from the laboratory dynamic triaxial soils tests, the numbers of axle loadings have been estimated for each settlement plate site. This information is given in the Table of Figure No. 4.

Site No.	Mile	Subdivision	MGT/Yr. (1980)	% Axles >30 Tons*	No. of Axles/Yr. Million		
					Light	Medium	Heavy
11	257.8	Wainwright	36.8	20	1.11	0.45	0.39
14	9.67	Edson	25.2	20	0.76	0.31	0.27
15	38.7	Edson	50.4	20	1.52	0.62	0.54
16	123.4	Vegreville	12.5	15	0.45	0.16	0.11
17		Calder Bypass	-				
18,19		Joliette	10.9	4	-0.53	-0.15	~.03

* Estimated for 1981 (when track settlements were recorded).

FIGURE NO. 4 - ESTIMATED AXLE LOADINGS AT
TRACK SETTLEMENT MEASUREMENT SITES

It is to be noted that the Table lists numbers of axles in each loading range. From track measurements, it has been documented that the subgrade remains loaded between axles of a freight car truck, and that it is more realistic to consider 1 loading cycle per truck (pair of axles). McGill Geotechnical Research Centre has utilized a simulated truck loading pulse in their dynamic triaxial soil testing program. In order to obtain the number of truck loading cycles applied in the field, the numbers of axle loads given in Figure No. 4 must be divided by two.

What are the traffic projections for the future? Annual tonnage is increasing, greater percentages of 100-ton cars are being added to the fleet, and older 40- and 50-ton cars are being retired. By the year 2000, it has been projected that tonnages in the Mountain Region could range from 76 to 92 MGT and percentage of heavy axles could go as high as 55 (the maximum 55 percent represents fully loaded operations in one direction and empty return movements, i.e. captive equipment unit train operations). Double trackage is being laid to handle the traffic, and when this is completed, the tonnage per track will be in the order of 38 to 46 MGT.

Suppose one made the assumption that traffic of the future could be represented as 60 MGT per track and 40% heavy axles. This translates into the following numbers of axles per year.

Light Axles	0-20 Tons	0.81 Million/Year
Medium Axles	20-30 Tons	0.58
Heavy Axles	>30 Tons	0.92
Total Axles		2.31

It may be noted that the total of 2.31 million axles per year is less than the 2.50 million carried over the Rivers Subdivision in 1980, but the number of heavy axles has tripled.

APPENDIX E

SUBBALLAST THICKNESS STUDY - LITERATURE REVIEW

SUBBALLAST THICKNESS STUDY - LITERATURE REVIEW

1. Stresses Beneath a Railway Track

D.L. Heath, M. Cothram

British Railways Research Department, Derby

The Railway Gazette, December 16, 1966

Load cell tests have confirmed the validity of stress calculations for common roadbed materials.

Research work was initiated by the Chief Civil Engineer of the British Railways and received international sponsorship in 1963 through the O.R.E. Bureau of U.I.C. (research program adopted by D.71 Specialist Committee). The overall aim of the complete investigation was to evolve a comprehensive design method for track foundations.

Beam-on-elastic foundation theory was used to determine tie pressure distributions and Boussinesq theory was utilized to derive vertical stress with depth curves. Stress curves were normalized with respect to tieplate loads. The study showed that:

1. The type of blanket material (subballast) did not appear to have an influence upon the level of stress induced within the formation (subgrade) beneath them.
2. There was close agreement between measured and theoretical stress.

It was concluded that a theory had been evolved which was of sufficient generality and accuracy for use in the solution of all track foundation problems. No soils property test results were given and no granular depth design charts were presented.

2. Track Foundation Design

J.M. Waters, M.J. Shenton

Research Department, British Railways

The Railway Gazette, October 4, 1968

A design procedure has been evolved for relating the depth between sleeper and subgrade to the axle loading and traffic speed.

Dynamic triaxial soils tests on London clay established a soil threshold stress level of approximately 7.5 psi. A depth of construction of 46 in. was shown to be required for an axle load of 15 tons travelling at a speed of 60 mph if the threshold stress level was not to be exceeded by more than 0.5% of the design load axles.

The design procedure proposed could be utilized to provide basic information from which the effects of proposed variations in the axle load, speed, and maintenance standards could be assessed.

3. Design of Conventional Rail Track Foundations

D.L. Heath, M.J. Shenton, R.W. Sparrow, J.M. Waters

British Railways Board Research Department, Derby

Institution of Civil Engineers, Proceedings Volume 51, February 1972

From results of repeated triaxial loading tests on cohesive soils, a level of stress was found above which repeated applications of load caused large permanent deformations and below which permanent deformations were small and terminating. Based on these findings a method of track foundation design was developed in which the depth of granular required to prevent excessive deformation of the subgrade could be predicted from the results of a simple laboratory repeated load test and a knowledge of the traffic loading to be carried.

Accepting that the design procedure should be based on a knowledge of both the variability of the applied loadings and material properties of the substructure, two approaches may be considered:

- a) Elastic approach based on mean loading and relatively frequent overstressing.
- b) Plastic approach based on extreme loading and very few overstressing events.

For the elastic approach, the following information is required:

- a) The applied loading spectra in terms of amplitude and frequency.
- b) The stress or strain distribution through the complete track structure.
- c) The relevant subsoil material parameters.

From the work it was found that:

- a) There was a linear relationship between tie loading and subsoil stress which was independent of speed and wheel arrangement.
- b) The scatter in stress levels between nominally identical positions in track substructure was highly dependent upon the ballast condition; hand tamping produced the highest concentration of pressure and mechanized tamping the highest scatter.
- c) For practical purposes, similar vertical subgrade stress distributions were obtained for both wood and concrete ties.
- d) In terms of vertical subgrade stress per unit tie load, sleeper spacing in range 25 to 31 inches had a negligible influence.
- e) Vertical stress distribution at the subgrade surface became practically uniform at a thickness of construction greater than 24 inches.
- f) Single layer elastic theory used with a simplified tie contact pressure distribution provided an adequate means of calculating subgrade vertical stresses - a more rigorous approach did not appear justified.

- g) Speed effects were negligible for both jointed and welded rail - absolute stresses varied with speed due to influence of speed on dynamic axle load.

Having concluded that mean vertical stresses in the subgrade could be predicted with reasonable accuracy by simple elastic theory, three statistical distributions were required for determining absolute stresses in the subgrade.

- a) Relationship between tie unit reaction and subgrade stress due to variability of ballast and subgrade.
- b) Relationship between unit wheel load and tie reaction due to variation in sleeper support conditions.
- c) The spectrum of actual dynamic wheel loads.

Author's Note: In the CN Rail Research study, statistical data has been developed on tie loads, dynamic wheel loads, roadbed elastic deflections with depth, subgrade soil surface strains, subgrade permanent settlement rates.

Dynamic triaxial soils tests using square wave loading pulses at 0.5 cps (pulse loading applied only on major axis principal stress, minor radial principal stresses held constant) were conducted on drained samples of London clay.

It was found that:

- a) Below roughly 7.5 psi deviator stress ($\sigma_1 - \sigma_3$), rate of deformation was reducing toward stable condition.
- b) An elastic strain of 0.65% represented a threshold level between continuing and terminating cumulative strain (10% cumulative strain was used as convenient figure for defining failure).
- c) Threshold stress was approximately a linear function of effective ambient pressure.

Depth of construction design charts correlating soil threshold stress with design axle loads were presented. For a 33-ton axle load and an assumed threshold stress of 7.5 psi for London clay, a depth of granular of 42 inches would be required.

Subgrade settlement rates were recorded at a number of British Rail test sites, and information effectively confirmed the validity of the threshold stress design method. Settlement rates as high as 100 mm/million axles were recorded where granular depths were 8 in. less than design figure, settlement rates as low as 5 mm/million axles were recorded where granular depths were 8 in. greater than the design figure.

Author's Note: The British Rail axle loading spectra was not presented, and it is not known how much time or tonnage one million axles represents. On the CN B.C. South Line, 50 Million Gross Tons (MGT) per year would represent roughly 2.5 million axles per year. In 1976, 26% or 0.65 million axles per year carried loads of 30 tons or greater. See Appendix D for characterization of CN railroad track vertical loading.

Some of the conclusions from the study:

- a) Even in the case of extremely conservative design, some non-zero rate of subgrade deformation must be accepted due to occasional overloads.
- b) The design method took no account of the axle load distribution within the total number of axles.
- c) The simple design procedure nevertheless produced construction depths apparently close to the optimum.
- d) The design procedure did not take into account the quality of ballast or subballast. It has been assumed that there was 12 inches of clean track quality ballast, and that the subballast complied with the inverted filter grading requirements specified for the blanketing function.
- e) The simplified procedure at present available provided a satisfactory and realistic technique for practical engineering purposes.

Author's Note: It was the intent of the current CN Rail Research study to investigate the validity of the British Rail design procedure in terms of North American traffic conditions and Canadian subgrade soil types.

4. Unpublished Document - Thurber Consultants Ltd.
Report on roadbed construction requirements for Edmonton Transit Light Rail Train (LRT)

Laboratory soils tests were conducted on samples of Lake Edmonton clay. The Edmonton LRT is underlain by Lake Edmonton clay which is covered by up to nine feet of various old fill materials. Based on performance of existing C.N.R. adjacent rail lines, the record would suggest that a suitable depth of ballast and subballast would appear to be in the range of 3 to 4 feet. The average undisturbed compressive strength of the Lake clay is 3226 psf (22.4 psi) with a range from 1448 to 6010 psf (10 to 41.8 psi). As the clay was shown to be susceptible to softening when exposed to a small flow of water, as a first approximation the undisturbed strength should be reduced by half. Cyclic load

tests suggested that the clay would behave in an elastic manner when the working stress level was 50 percent or less of the maximum or failure stress. Therefore, a first approximation of the working or "threshold" stress of the Lake clay when reduced for softening and repetitive loading is 800 psf (5.56 psi). This was based on very limited number of tests.

Simple elastic calculation using the classic Boussinesq method was carried out to produce an estimate of the stress induced in the subgrade by the Rapid Transit axle loading of 14.25 tons. Using the reduced working compressive stress for the Lake clay of 800 psf and conventional static bearing capacity of 680 psf, the vertical stress chart showed that a 22-inch thickness of ballast and subballast was required to keep stresses in the clay within limits.

Using the British Railways design chart and the Lake Edmonton clay "threshold" stress of 800 psf, a required depth of ballast and subballast of 29 to 30 inches was indicated.

Because of the moderate number of test results available, and to make the best agreement with present C.N.R. experience, a total depth of ballast and subballast of 36 in. was recommended.

5. Stress Distribution in Track Structure

Dr. R.M. Hardy, Dean of Engineering, University of Alberta
AREA Bulletin 614, Proceedings Volume 69, June-July 1968

A study was made on the Sangudo Subdivision in the Mountain Region of the Canadian National Railways in western Canada. It is characteristic of all soils in the general area that the clay fractions have significant percentages of montmorillonite clay mineral. These soils therefore all have swelling tendencies in varying degrees. Severe deterioration of roadbed occurred following introduction in late 1962 of 100-ton sulphur car and 6-axle locomotive traffic.

On the basis of soil strength and soil shearing stresses induced by E-40 and E-66 loading, the following granular thicknesses were recommended for the upgrading program:

- a) Mile 0-60, for the till soils - 18 inches.
- b) Mile 60-103, for the clay soils - 24 inches.
- c) For muskeg areas - 36 inches, with fill side sloped not steeper than 3:1.

Measurements of rail (60 and 100 lb.) and subgrade static deflections were made under E-11, E-40, and E-66 loading. The deflections in the subgrade did not exceed 0.15 inches (for E-66 loading), and this value was only some 20% of the maximum rail deflection (0.75 in.).

The subgrade was relatively stiff as compared to the loose ballast materials, and the weight of rail and ballast thickness (9, 18 and 30 in.) had comparatively small influence on the rail and subgrade deflections. The predominant factor influencing the subgrade deflection was

the wheel load, deflections roughly doubling in going from E-40 to E-66 loading, irrespective of the ballast thickness.

It was concluded that this did not mean there were no benefits to be gained by increasing the weight of rail and thickness of ballast. If the subgrade soils are of types susceptible to strength deterioration by straining, then a new cycle of subgrade troubles can be initiated by the increased wheel loads, which will be largely independent of upgrading of the other track structure elements. In many locations, upgrading of the subsoil conditions is therefore just as necessary as the improvement of other elements in the track structure.

6. An Analytical and Experimental Evaluation of Concrete Crosstie and Fastener Loads
Report No. FRA/ORD-77/71 December 1977 Interim Report
Report prepared for U.S. Department of Transportation
Federal Railroad Administration, Office of Research and Development, Washington, D.C. 20590

This large report contains a tremendous amount of information, not the least of which is a review of track analysis models. Some 25 theoretical models have been listed, and many of the advantages and disadvantages of each have been pointed out. There appears to be no lack of theory but there is a lack of good track-measured data relating roadbed granular depth to track maintenance requirements.

7. Ballast and Foundation Materials Research Program
Technical Data Bases Report
Q.L. Robnett, M.R. Thompson, W.W. Hay
Department of Civil Engineering, University of Illinois
Report prepared for Association of American Railroads, Chicago.
July, 1975.

This report presents an excellent summary of current technology relative to the following:

- a) Procedures for evaluating ballast and subgrade material properties.
- b) Factors which influence ballast and soil material properties.
- c) Relations between ballast and soils material properties and track system behaviour and performance.
- d) Applicability of structural analysis models to predicting behaviour of track systems.
- e) Transfer functions relating track behaviour to performance.

Based on the fairly comprehensive information and literature reviewed, the following conclusions were drawn:

- a) Ballast and subgrade soil materials receive inadequate consideration in the analysis and design of conventional track support structures.

- b) Strength properties of ballast and roadbed soil materials are rarely determined.
- c) Although numerous quality tests are normally conducted on ballast materials, few correlations have been developed between quality test results and field performance of ballast materials.
- d) Stress-strain characteristics of ballast and subgrade soil materials should be evaluated using testing procedures which closely simulate actual in-service loading conditions. It appears that the repeated load triaxial test can be used to adequately evaluate the "strength" characteristics of ballast and subgrade materials. This should include determination of resilient and permanent deformation behaviour under repeated loading.
- e) It is necessary to establish the magnitude and range of all such factors which significantly affect the repeated load response of ballast and subgrade soil materials.
- f) "Structural models" presently available for structural analysis of the track support system do not adequately consider the "stress dependent" nature of ballast and subgrade soils materials.

8. Railroad Subgrade Stresses

Herbert O. Ireland, Professor of Civil Engineering
University of Illinois at Urbana-Champaign
Bulletin 641 - American Railway Engineering Association

If it is assumed that the ballast and subgrade below ties is completely elastic, the distribution of vertical pressure beneath the ties can be determined using Newmark's graphical solution of the Boussinesq theory. A graph relating vertical stress to depth below tie base has been prepared for the following assumed conditions:

- a) Vehicle with axle load of 32.2 tons.
- b) Axle load supported by 3 ties.
- c) Each rail would load the outer one-third of each tie.
- d) Ties are 8'-6" long, 9 inches wide, 6 inches thick, and spaced at 20 inches center to center.

To prevent bearing capacity failure or undue creep beneath a loaded area, the level of stress in the subsoil should not exceed an allowable bearing pressure based upon a suitable factor of safety. A safety factor of at least 2 was recommended, and under no circumstances should the factor of safety be less than 1.5 unless a certain amount of creep is acceptable. The allowable bearing pressure of a cohesive subgrade may be taken as $5 C/F$ where C is the undrained shear strength (cohesion) of the soil and F is the factor of safety. Ideally, the saturated undrained strength should be used because this probably represents the minimum strength that is likely to exist in the field and therefore should be the basis for design.

Design curves of required granular thickness for various factors of safety with respect to subgrade cohesion strength have been presented.

The sample chart was based on static loads only and did not consider dynamic effects.

9. Railroad Track Support Granular Design Depth Guide

This guide prescribes suggested requirements for all-weather track support granular fill depths for use on new construction designed to carry 300 kN (33-ton) axle loads on standard gauge track constructed to current (1978) North American Standards. The granular design depths, related to soils designations as adopted by the Unified Soil Classification System, are based on a paper by G.P. Raymond, "Design for Railroad Ballast and Subgrade Support", Journal of the Geotechnical Engineering Division, Proceedings, American Society of Civil Engineers, Volume 104, No. GT1, Paper 13471, January 1978, pp. 45-60.

For unlimited yearly tonnage (MGT) and speeds to 100 mph, the recommended ballast depth below tie base is 14 inches and the recommended subballast depth is 16 inches for all subgrades other than groups GW and GP (gravels and gravel-sand mixtures). For new track formation, compact subgrade to 95%, D 698.

While this literature review has by no means been comprehensive, it will have served to highlight some of the factors involved in roadbed design. Particular attention has been paid to the British Rail design method because it was one of the objectives of this current study to evaluate its validity in terms of Canadian subgrade and rail traffic conditions.